FOUNDATIONS OF BRIDGES AND BUILDINGS

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FOUNDATIONS OF BRIDGES AND BUILDINGS

CHAPTER I

TIMBER PILES AND DRIVERS

Art. 1. Foundations

A structure usually consists of two parts, one of which is supported by the other; the upper part being known as the superstructure and the lower part as the substructure. In a bridge the superstructure is composed of the beams, girders, or trusses, together with the floor system and bracing which they carry; while the substructure consists of the piers and abutments, including their supporting bases.

The substructure frequently consists of two parts which differ more or less in form and character, the lower part being called the foundation which supports the rest of the entire structure. Sometimes the term foundation is used without regard to any substructure; as, for example, when it is applied to the independent structure which supports a machine.

The foundation of a structure may then be defined as that part of it which is usually placed below the surface of the ground and which distributes the load upon the earth beneath.

Foundations are divided into various classes. The simplest form is obtained by widening merely the base of a wall or pier, so as to distribute the load over a sufficient area on the foundation bed of earth. Another form is known as the spread footing, in which the bearing area is enlarged, either by reinforcing the concrete base with steel bars or by inserting one or more tiers of steel beams.

Pile foundations consist of a base of concrete or of timber grillage, supported by piles which distribute the load to the earth through a considerable depth, either by friction alone or by friction combined with bearing on the ends of the piles.

When the bottom of the foundation has to be located on a bed of hard material at a considerable depth below the surface of the ground, the classes of foundations are distinguished by the respective methods required to sink them into position.

Foundations built in open wells are used when the excavation can be made either in the dry or with no more interference by water than may be controlled by a reasonable amount of pumping.

When open caissons are employed, the excavation is made through the water under ordinary atmospheric conditions, and after the bottom is sealed by concrete the rest of the foundation is built in the open air.

Pneumatic foundations are those in which the excavation is made by working in compressed air in the chamber of a caisson, on the roof of which the concrete or masonry is built up in the open air during the operation of sinking.

Many kinds of foundations also require the use of a temporary structure known as a cofferdam in order to exclude the water from the site of the foundation during its construction.

The character of the earth at the site, extending down to the bed on which it is to be founded, and the influence of water, if any, determine the kind of foundation to be employed in any given case; with due regard, however, to economic limitations.

These general classes of foundations and their subdivisions will be described and illustrated in the subsequent chapters of this volume, together with the general methods of placing them in position. Occasional notes on some of the special equipment required will also be given.

ART. 2. CLASSIFICATION OF PILES

A pile is an element of construction placed in the ground, either vertically or nearly so, to increase its power to sustain the weight of a structure, or to resist a lateral force.

Piles are designated by the material of which they are composed; as, for example, timber piles; by their form of cross-section, as round or octagonal piles; by their inclination, as batter piles; by their use, as guide piles, sheet piles or fender piles; or by some attachment to their feet in order to increase their bearing power, as screw piles or disk piles.

A bearing pile is one which carries a superimposed load. Its form of cross-section depends upon the material of which it is composed, and may be round or circular, square, octagonal or annular. Its longitudinal section is frequently tapering, but sometimes its cross-section remains uniform throughout the length of the pile.

The head of a pile is its upper end; the foot of a pile is its lower end; the butt of a pile is its larger end; the tip of a pile is its smaller end. These definitions show that the terms head and foot relate to the pile in its final position only, while the terms butt and tip apply to a tapered pile either before or after it is placed in position.

The term "top" is often ~~applide~~ to one end of a pile but this is ambiguous, since the upper end of the pile in the tree may be either the upper or the lower end after the pile is driven; its use should therefore be discouraged. The same objection holds with respect to the term "point," which is often used to designate the small end of the pile, which may be either pointed or left blunt by cutting off the end perpendicular to the axis of the pile.

A batter pile is one driven at an inclination to resist forces which are not vertical. They are sometimes called spur piles. When a pile structure is built to resist lateral pressure, experience has proved the importance of relying chiefly upon batter piles, rather than upon the cross-bracing of vertical piles, to insure lateral stability. When piles are employed to resist the lateral pressure of earth and to form a wall which is intended to be water-tight, they are called sheet piles. Their form usually differs from that of other piles, there being a considerable variety in their cross-sections both for timber as well as steel sheet piles. The subject of sheet-piling is discussed in Chap. V

and various applications are given in subsequent chapters. Short piles are sometimes driven in order to compress and consolidate the ground over a considerable area to increase its bearing power, but usually this result is more economically attained by means of sand piles, perhaps combined with sand or cinder filling on top.

Bearing piles are used in foundation construction under two typical conditions: first, when the piles are driven through soft or fluid material into or to a stratum of firm or practically unyielding material; second, when no hard bottom can be reached by any reasonable length of pile and the friction of the pile in the ground is sufficient to support the load with safety.

In the first case, the pile receives little if any lateral support and therefore acts as a column; while in the second case, the true pile action occurs and the load is either limited by the adhesion of the ground to the surface of the pile or the compressive resistance of the material in the upper part of the pile.

Bearing piles located in streams often have to resist lateral forces due to the impact of drift, ice, etc. As far as possible such forces should be provided for by sway or lateral bracing.

The most favorable condition for the use of bearing piles occurs when a firm stratum can be reached by piles of ordinary dimensions, and therefore easily obtainable in the markets, and the overlying material is compressible, so as to be readily penetrated by piles, but sufficiently compact to prevent the piles from bending and lateral displacement.

Guide piles are used to support the horizontal timbers or wales which, in turn, guide and support the vertical sheet-piling. Their principal application occurs in cofferdam construction (see Art. 69), but they are also used in ferry slips, and to aid in locating and sinking open and pneumatic caissons in streams or lakes.

Fender piles, as their name implies, are driven at wharfs or in front of large masonry structures or other important works, to protect them from sudden blows by vessels. In addition to the uses of piles mentioned above, they are employed in dikes, jetties, and other structures.

Timber piles are very extensively employed in railroad construction and maintenance, for trestle bridges, either as temporary structures until the filling in of embankments or more permanent bridges of steel or concrete can be built to replace them, or until they are reconstructed of the same material. Trestle bridges with pile foundations are generally built in emergencies resulting from washouts, fire or accidents of any kind. This is due to the rapidity with which the pile foundation can be put in place and the rest of the structure built, piles and large dimension timbers for framing being regularly carried in stock. Guide piles, fender piles and, in fact, all piles used in temporary structures are likewise composed of wood.

It will hence be noted that piles are very extensively used in modern engineering construction. While it is certain that timber piles were known as long ago as the early lake dwellers of Europe, they have been used continuously since that time, for foundation purposes. On the contrary, concrete piles were introduced in the opening years of the twentieth century. Metal piles were first used in 1838.

The materials employed for piles include wood, concrete (either plain or reinforced), cast iron, wrought iron, steel and sand. Sometimes two materials are used in combination; as, for example, in a wooden pile surrounded by a protection of reinforced concrete, or in a hollow metal pile filled with concrete, either with or without reinforcement. Piles composed of sand are made in place in the earth in a vertical cavity formed for the purpose and hence serve chiefly to compact the earth and thereby increase its bearing power.

In practice, a pile is usually placed in position in the ground by driving it with a steam-hammer or a drop-hammer, either with or without the aid of one or more water-jets. In rare instances, a pile may be sunk in place by static pressure, either by means of block and tackle or a weight of some kind. Sand piles or certain types of concrete piles are, however, cast directly in place.

The principal use of piles occurs in the foundations of bridges, buildings and other structures, in which they act simply as bearing piles.

ART. 3. TIMBER PILES

In the specifications for timber piles adopted in 1909 by the American Railway Engineering Association, the following kinds of wood are included for piles intended for standard construction purposes and designated as "railroad heart grade": white, burr, and post oak; longleaf pine; Douglas fir; tamarack; eastern white and red cedar; chestnut; western cedar; redwood and cypress. For temporary construction, the following kinds of wood are included for piles designated as "railroad falsework grade". red and all other oaks, not included in railroad heart grade; sycamore; sweet, black and tupelo gum; maple; elm; hickory; Norway pine or any sound timber that will stand driving.

The principal difference between these two grades relates to durability, although the former includes several of the most valuable species of wood used in modern engineering construction, as longleaf yellow pine, Douglas fir and white oak. Cedar piles are noted for their long life or durability. Although spruce is not specifically mentioned in these specifications, spruce piles are extensively used, especially in New England, both for railroad structures and other buildings. Spruce from certain localities has unusual toughness, giving the piles increased resistance to the tendency to split and broom when driven.

Among other species which have been used to a very limited extent for piles may be mentioned beech, ash and basswood. In Florida, palmetto piles are used, as this wood is comparatively free from attacks of marine borers, known as the Teredo. White pine piles were used in the northern central states before the close of the nineteenth century, but since then this species has become too valuable on account of its demand for other uses in building construction. Yellow pine, Douglas fir, spruce, cedar and other conifers have increased value for piles because they are so straight and free from large branches, and can be obtained in greater lengths. The longest piles used in single sticks are Douglas fir. Oak piles are hard and tough, but are not so straight and smooth and have the added disadvantages on account of weight, of increased cost of

transportation and of liability to sink in water unless lighter logs are used in rafts to buoy them up.

The specifications of the American Railway Engineering Association also include the following requirements, for the railroad heart grade: Piles shall be cut from sound trees; shall be close-grained and solid, free from defects, such as injurious ring shakes, large and unsound or loose knots, decay or other defects, which may materially impair their strength or durability. Piles must be cut above the ground swell and have a uniform taper from butt to tip. Short bends will not be allowed. A line drawn from the center of the butt to the center of the tip shall lie within the body of the pile. Unless otherwise allowed, piles must be cut when the sap is down. Piles must be peeled soon after cutting. All knots shall be trimmed close to the body of the pile. Square piles shall show at least 80 percent heart on each side at any cross-section of the stick, and all round piles shall show at least a 10½-inch diameter of heart at the butt. Piles of the railroad falsework grade, however, need not be peeled, and no limits are specified as to the diameter or proportion of heart. These specifications as revised, from time to time, are published in the Manual of the American Railway Engineering Association.

The provision regarding the lateral curvature of a pile is modified by some engineers so that the center of any cross-section shall not depart more than one-eighth of its diameter from the straight line joining the centers of the butt and tip. In another specification, this distance is made 1 percent of the length of the pile. When a pile has bends in two directions, it is regarded as a sufficient cause for rejection on first-class work. It has been found by experience that spruce piles selected for their straightness and smoothness could be driven satisfactorily where it was impossible to drive oak piles, which were irregular in shape and covered with knots. Timber piles are driven with the butt down under some conditions, this topic being discussed in Art. 15.

The time of year in which timber is cut for piles does not receive the degree of attention which it deserves. It affects

both the strength of the timber and its durability. Tests made in Germany of four spruce trees, growing close together in the same soil, showed that if the strength, when cut in December, is taken as 100 percent, those cut in January, February and March had strengths of 88, 80 and 62 percent respectively. "Beech timber cut in December and January gave an average mechanical life of six years, whereas the same kind of timber cut in the same location in February and March gave a service of only two years."

Experience in this country has also shown conclusively that the use of piles of the best species of wood may lead to serious loss when it is cut in the summer and left only a short time before the bark is peeled. Decay due to fungi and the ravages of worms, which became manifest when the sapwood began to decay, required, in one case involving a very large number of piles, the replacement of the whole lot within four years, some of them being eaten through entirely within two years.[1]

Foundation piles when cut off below the ground-water level, apparently have an indefinite life. For example, in reconstructing a bridge, timber piles were removed which indicated no material decay after being in service 600 years. A still more conspicuous example was brought to the attention of engineers and architects, when the Campanile of St. Mark's in Venice fell in 1902. The piles in the foundation, which had been in service for 1002 years, were found to be in such a good state of preservation that they were allowed to remain to support the reconstructed tower.

A lagged pile has pieces of timber bolted around the sides of the pile, in order to increase its bearing power. It increases the area of cross-section and also the surface of the sides of the pile which is of more importance, since such piles are used only in very soft material. The New York City Dock Department made a test in 1902 of the relative bearing power of lagged and unlagged piles driven in North River mud, the results of which are recorded in the Transactions of the American Society of

[1] See Railroad Gazette, vol 31, page 865, Dec 15, 1899.

Civil Engineers.[1] The discussion by the author of the paper implies that the ultimate bearing power was increased about 50 percent. The total penetration of the piles ranged from 47.1 to 52.6 feet, while the lagging was only 30 feet in length. Although it is not stated what position vertically the lagging occupied, it appears that the surface in contact with the mud was increased about 70 percent.

In some tests made in 1915 with a cluster of piles spaced 2½-feet centers and with alternate piles lagged, the settlement under a total load of 230 tons was 11¼ inches in 280 days, while for a cluster in which all the piles were lagged and spaced 3¼ feet in one direction and 3½ feet in the other the settlement was 8¼ inches. For the first 60 days the settlement was about the same for both clusters. The lagging consisted of four 5- by 6-inch sticks 30 feet long, spaced around the four quarters of the circumference and at the lower end of the pile. The total embedment of the piles below ground was 65 feet.

ART. 4. FORM AND DIMENSIONS

Since a timber pile generally consists of the lower portion of the trunk of a tree, after its branches and bark are removed, and the knots trimmed close to the body, its cross-section is round or approximately circular. Square piles are rarely used as bearing piles, and only to a limited extent for special purposes, one of which is to form large timber sheet piles by the addition of scantlings on two sides to form tongue-and-groove joints. Since the introduction of steel sheet-piling, there is but little need for framing sheet piles out of 12- by 12-inch, or even larger, timbers (see Art. 61).

The specifications referred to at the beginning of the preceding article contain the following paragraph, relating to dimensions: For round piles, the minimum diameter at the tip shall be 9 inches for lengths not exceeding 30 feet; 8 inches for lengths over 30 feet but not exceeding 50 feet; and 7 inches for lengths

[1] Vol. 54 F, pages 8 and 27, 1905.

over 50 feet. The minimum diameter at one-quarter of the length from the butt shall be 12 inches, and the maximum diameter at the butt 20 inches. The same requirements apply to the square pile, by substituting thickness for diameter.

The relation between the diameters of butt and tip depends upon the length of a pile and naturally varies for different species of wood. The diameters of piles for ordinary buildings are usually somewhat smaller than for bridges and very heavy buildings, but the diameter of tip should not be less than 6 inches in any case. When a pile acts principally as a column, it should have a larger tip than if its resistance depends mainly on friction.

The clearance between the leads of pile-drivers, and between which piles must be placed to drive them, is ordinarily 22 inches, and it will be noted that the maximum limit placed upon the diameter of butt, in the specifications quoted above, is 2 inches less. It may be stated that the diameter of butt usually ranges from 11 to 16 inches in foundations which are intended neither for very light nor for exceptionally heavy structures.

The length of a pile necessarily depends upon the character of the earth into which it is to be driven. Piles as short as 10 feet have been used, but it is questionable whether this is not too low a minimum. In ordinary construction, the length of piles varies roughly from 20 to 40 feet. As an illustration of the use of long piles, Douglas fir piles ranging in length from 60 to 120 feet were driven in 1907 for the trestle approaches of the Dumbarton bridge at San Francisco Bay, the penetrations in some cases being as great as 60 feet. The tip was not less than 9 inches, while the butt was limited to 22 inches. In the jetty construction at the mouth of the Columbia River piles 130 feet long were driven 50 feet into the bed of the river; they were 30 inches in diameter at the butt. Even greater lengths up to 175 feet were formerly used on the Pacific Coast, but since the best timber next to the coast or navigable streams has been cut, the available lengths are limited by the conditions of railroad transportation. Where the character of the earth or of the several strata to be penetrated is fairly uniform over the area of a

given site, it is desirable to use piles as nearly alike in diameter and length as can be secured economically in the available markets. Where the driving is easy, a small pile is frequently as advantageous as a large one; but where the driving is hard, a large pile is required so as to have the necessary strength and stiffness to stand the driving.

The principles of good design and economic construction require that the proper lengths of piles be determined in advance. In the absence of definite knowledge by previous pile-driving experience at the same location or contiguous to it, a careful exploration of the ground should be made by means of auger or wash-borings or by means of test piles. Tests should be made at certain intervals along the line of a trestle bridge, at the locations of piers and abutments, or at several places distributed over the area of a building foundation. Driving test piles is advantageous, since it furnishes information at the same time on the number of blows required to secure the necessary total penetration and, hence, the approximate time for the subsequent work. This item alone is frequently worth far more than the cost of the investigation. On the other hand, emergencies may arise in which the value of such preliminary tests in saving material and labor in construction may be offset by a greater loss due to delay in resuming traffic operations. Methods of making explorations by different appliances are described in Chap. XVII.

If some other method is used to determine the supporting power of the earth, and it is proposed to compute the size of pile, it is well to consider that "with the usual methods in vogue, n which large initial stresses are to be expected, it is not safe to use piles of diameters which would be just large enough to support the developed supporting power of the earth, nor would it be practicable to secure or drive them."

A convenient table prepared by E. O. Faulkner, for calculating the volume of piling in cubic feet, and which is based on the prismoidal formula, may be found in Engineering News, vol. 54, page 170, or in Richey's "Building Foreman's Pocket-book."

ART. 5. THE PHENOMENA OF PILE DRIVING

The term "pile driving" is applied to the operation of taking a pile and forcing it into a definite position in the ground without previous excavation. A number of methods are employed for this purpose, which require different kinds of equipment. Historically, the oldest method of driving a pile is by means of a hammer. While very small bearing piles, or posts, were doubtless driven at first by hand with a maul or beetle, those of larger size, usually designated as piles, required the use of a machine by which a hammer was raised with the aid of a pulley and rope and allowed to drop on the head of the pile. A weight used in this manner was hence called a drop-hammer. At first men, then horses, and afterward the steam engine were used to raise the hammer.

After the invention of the steam engine, steam-hammers were designed in which the driving weight is lifted a short distance by steam pressure and allowed to fall by gravity, the rapidity of action being greatly increased. Subsequently, steam-hammers were invented in which steam pressure reinforces the action of gravity on the down stroke. At one time pressure due to the explosion of gunpowder was used to drive piles, but that method is now regarded as antiquated. To a very limited extent pile driving has been accomplished by placing a static weight upon a pile and rocking it to and fro in soft ground, to which condition this method is practically limited.

Another method of more recent discovery, which has greatly advanced the art of pile driving, consists in the use of the water-jet to aid in displacing the earth at the foot of the pile and to lessen the friction of the pile as it descends through the surrounding material. This method is generally employed in conjunction with the use of a hammer, although occasionally the hammer may serve merely as a static weight during a portion of the time required to sink the pile.

The phenomena of pile driving may perhaps be most readily understood by the student by considering the case in which a timber pile is driven vertically into the ground by means of a

drop-hammer. After the piles are delivered on the site within reach of one of the lines of the pile-driver which is used to handle the piles, the line is made fast to a pile near its head and first dragged, if necessary, close to the front of the pile-driver, and then hoisted until it is suspended in the air. It is next placed and held laterally between the pair of tall parallel members of the pile-driver known as the leads and between which the hammer is guided in its movements. After lowering the pile until its foot rests on the ground, the line is released The hammer, being held at the top of the leads by the other line, is now released and in falling strikes the head of the pile. It is then raised again and released for the second blow, and so on for successive blows until the required penetration of the pile is obtained.

During its fall, the velocity of the hammer is accelerated until the instant when the hammer and the pile, in connection with a certain mass of earth beneath and around it, move together. When the hammer strikes the head of the pile the pressure between the pile and hammer increases from zero up to a certain value when the pile as a whole begins to move After all the compression in both hammer and pile has taken place, they will move together. Their velocity is then gradually reduced to zero by the varying resistance of the earth during the time of penetration for the pile. Some of the work done by the falling hammer is consumed in overcoming friction, in crushing and heating the head of the pile, and in compressing the pile and hammer, while the remainder causes the penetration of the pile

In careful experimental investigations conducted by ERNEST P. GOODRICH, with an apparatus designed to show the exact vertical motion of the pile, the time occupied by this motion, the velocity of the hammer as it strikes the pile, the velocity of the pile at each instant of its movement and the amount of compression suffered by the head of the pile from the blow of the hammer, it was found that on the average the penetration, measured from the deepest point, varies practically as the square of the time measured from the final instant. The autographic records showed also that, in the majority of cases, the

final magnitude of the force acting on the pile is the same as its initial magnitude when the pile and hammer move together; and prove conclusively that the hammer remains in contact with the pile until the motion of the latter has ceased.

Small-sized experiments on pressing sticks with blunt tips into sand and other kinds of earth, as well as observations of regular piles, show that a conical mass is formed at the tip and pushed along, while curved-flow lines of earth appear as the material is pushed aside and compressed. The extent of the movement depends upon the compressibility of the earth. Often some of the material near the sides of the pile will move upward slightly. It is thus seen that the supporting power of the ground penetrated is one of the elements which determines the load which a pile can bear. In most cases this supporting power of the ground increases more or less with the depth, and hence the load depends upon the total depth of penetration. Sometimes the larger part of the superimposed load is transmitted by the pile through its foot to a hard substratum, and therefore acts like a column. When the pile is supported entirely by the frictional resistance between its sides and the earth, the load is transmitted to a deep ground level in a conoid of pressure through the earth above it. Usually these two methods of transferring a load from a pile to the earth act together in varying proportions.

Art. 6. Pile-Drivers

A pile-driver is a machine for driving piles. Its characteristic feature consists of the leads, which are upright parallel members to support the sheaves used to hoist the hammer and piles, and to guide the hammer in its movement. The leads are held in position by being framed with back stays and other bracing into the form of a tower supported on horizontal sills. In a standard form of contractors' pile-driver the bed frame containing the sills is extended back far enough to support the hoisting engine and boiler, and the whole outfit is mounted on rollers as illustrated in Fig. 6*a*.

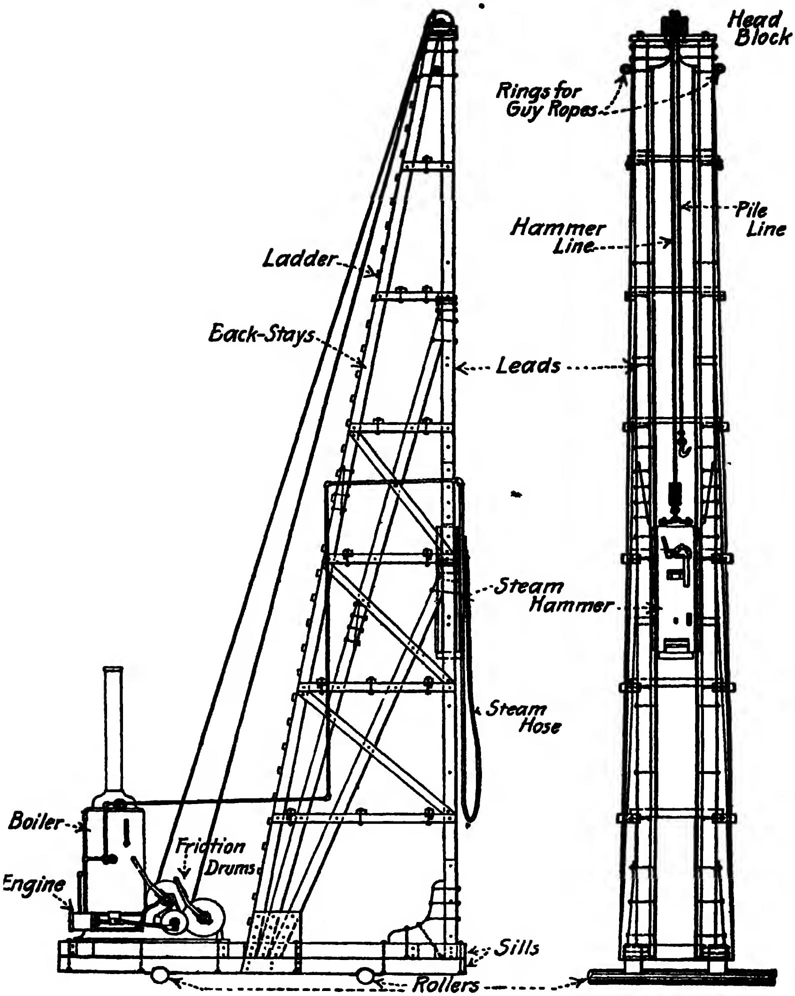


FIG. 6a.—A Standard Type of Contractor's Pile-Driver.

Pile-driver towers are constructed either of timber or of steel, and are built in a variety of forms for different purposes, or conditions. Rungs are attached to the rear inclined posts, or back stays of the tower, to form a ladder. The bracing consists of horizontal and diagonal members. In the figure two long diagonal braces are shown in addition to the diagonals in each panel. Sometimes the long diagonals are omitted and only short diagonals are placed in every panel, while in the smaller towers all diagonals may be omitted. Occasionally, the lower diagonals are extended over two panels, or long diagonals may be employed without any short ones. The tower is braced laterally either by guy ropes attached to the rings near its top or by long inclined posts, or wind braces, in which case the bed frame is generally widened to support these braces. Leads as long as 100 feet and 1 inch under the head-block have been built. (See also Fig. 64*a*.)

By the addition of roller bearings a driver may be moved forward, backward and sidewise. When it is mounted on a turntable, it is called a swiveling pile-driver, and combines swinging to the right or left with the motions noted in the previous sentence, the movement sidewise being made, however, by changing the rollers. The inner faces of wooden leads are protected by channel-iron liners, in order to reduce friction and wear.

A driver intended to be used in excavations to drive piles below the level of its supporting track, sometimes has rigid detachable leads which extend to the required depth. A better arrangement consists in the use of telescopic leads which slide inside of the stationary leads, and are handled by an extra line to a third hoisting drum. By this means piles may be driven without the aid of a follower in deep trenches or through contracted openings, or in the bottoms of cofferdams containing a large amount of internal bracing. Floating pile-drivers mounted on scows have had, in exceptional cases, 100-foot telescopic extension leads working within 100-foot fixed leads. With such equipment it has been possible to drive piles 35 to 40 feet below the water surface with the aid of a follower that was

thus guided at its lower end as well as at its upper one. Hanging leads, which can also be used for the same purpose as extension leads, are often used in connection with ordinary derricks or cranes.

ART. 7. TRACK PILE-DRIVERS

Pile-drivers of recent design for railroad service have been developed to a high degree of efficiency. In the Report of the Committee on Wooden Bridges and Trestles of the American Railway Engineering Association in 1911,[1] are contained the following:

DESIRABLE FEATURES OF A TRACK PILE-DRIVER

(1) Steam-hammer. To secure greater rapidity in driving and with less injury to the pile than that secured by the drop-hammer. (2) Water-jet apparatus. (3) Turntable allowing practically a complete rotation. In most cases the work can be done from either side, and in many of the remaining cases it is possible to foresee the nature of the work and to head the driver in the proper direction at the nearest Y or turntable. Sometimes, however, turning facilities may be far distant, or a pile-driver may be caught between two washouts when it becomes essential to be able to turn the machine to perform the work at both places. (4) Swinging leads. The leads require an efficient rigging to permit driving piles with a batter in either direction. When driving across the track on such work as driving bents for an adjacent track, it is convenient to be able to drive with the leads not fully raised, so as to secure the proper batter. (5) Self-propelling mechanism. The greater the tractive force and speed the more independent is the pile-driver from locomotive service. They should preferably be sufficient to dispense with a locomotive except for long hauls. (6) Restricted projection on the side opposite the leads when swung across the track and without unnecessary weight. (7) High-speed power service for raising the leads. On a main line it is frequently possible to drive only one or two piles before running

[1] See Proceedings, vol. 12, Part I, page 200.

to a siding. In some cases the character of this apparatus to raise the leads determines whether a single pile can be driven between trains or will delay a train. (8) Adequate overhang. To enable machines to drive piles as far ahead of the leading wheels and as far sidewise as possible. On work for double-tracking the sidewise reach should be sufficient to drive a bent on the new track from a position on the old track. (9) Facilities for driving below the track. (10) Ability to shift the hammer when the leads are down. (11) No obstructions in the view of the engineman and niggerhead operator. (12) Length of leads. To accommodate the longest piles practicable. (13) Strength and rigidity of supports for leads and hammer. They should be adequate to handle the hammer and the heaviest wooden pile without damage. It is now becoming important to be able to handle concrete piles. (14) Stability. The driver, while standing on its own wheels, without any jacks or special supports, should be able to pick up and drive a pile in any position within its reach. (15) Flush ends. For convenience of transportation in freight trains, no part projecting beyond the drawheads. Otherwise an idler is required which then may be used as a tool car. (16) No lengths of steam hose that might be replaced by pipe.

No single make or design of driver has incorporated every one of these desirable features. Those which come nearest to doing so are not of the combination type, but are designed especially for exclusive use as pile-drivers. In different makes the reach ranges from 15 to 21 feet ahead of the wheel base, the reach sideways from 20 to 33 feet from center of track, while the longer leads are from 40 to 47 feet. Some drivers are equipped with both steam- and drop-hammers and the best ones have a water-jet outfit. The turntable is usually on top of the car body, but in one case a hydraulic turntable is provided which takes bearing on the track, raises and turns the entire car (Fig. 7*a*). Those which are self-propelling have a speed from 8 to 25 miles per hour.

In one typical form the aim has been to combine the functions of a pile-driver with those of a steam-derrick car in the erection of small bridges, the maintenance of bridges and culverts, pull-

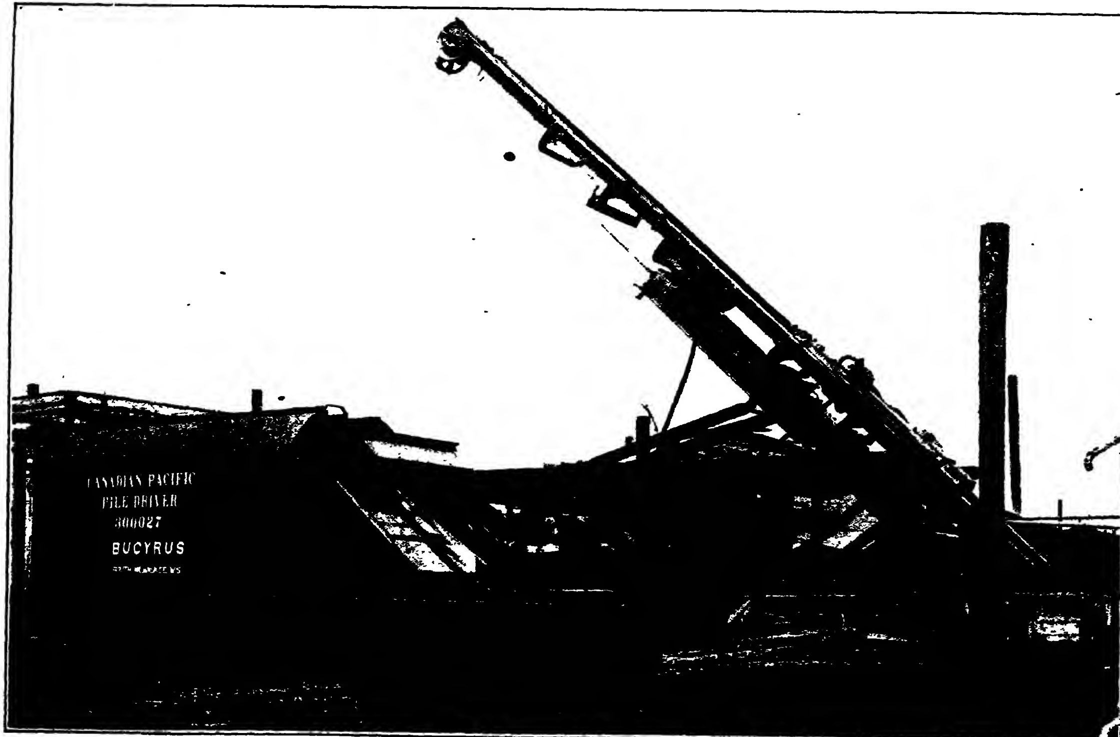
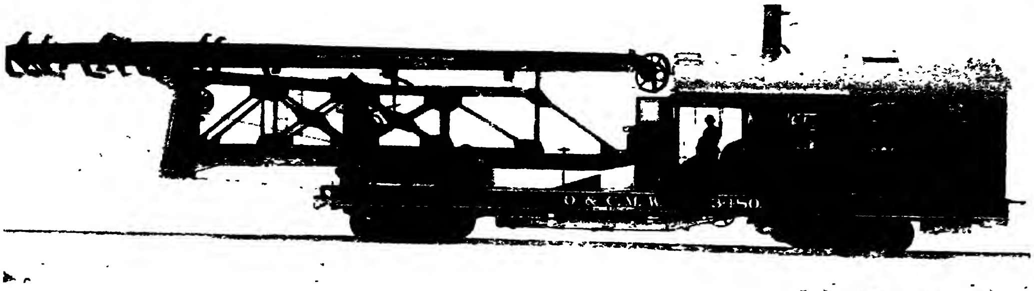


FIG. 7a.—Self-propelling Track Pile-Driver Rotated on its Turntable to Drive a Pile at its Extreme Reach from the Track. The leads are being lowered.

(*Facing p.* 18.)



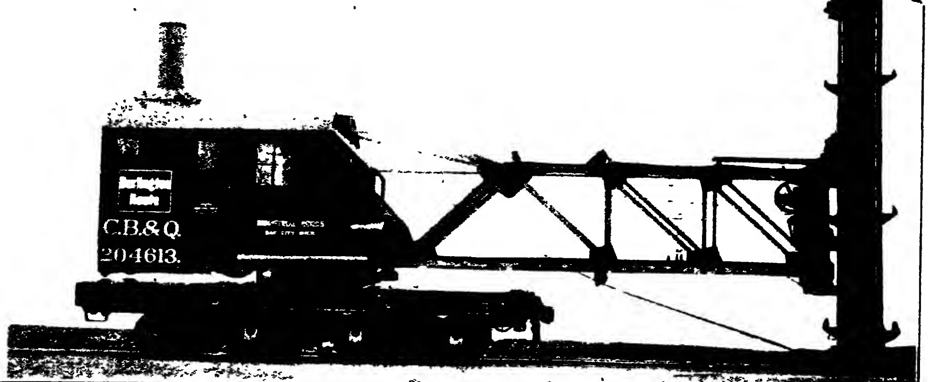


FIG. 7*b*.—Self-propelling Track Pile-Driver (Industrial Works) with Leads in Traveling Position.
FIG. 7*c*.—Convertible Crane Pile-Driver.

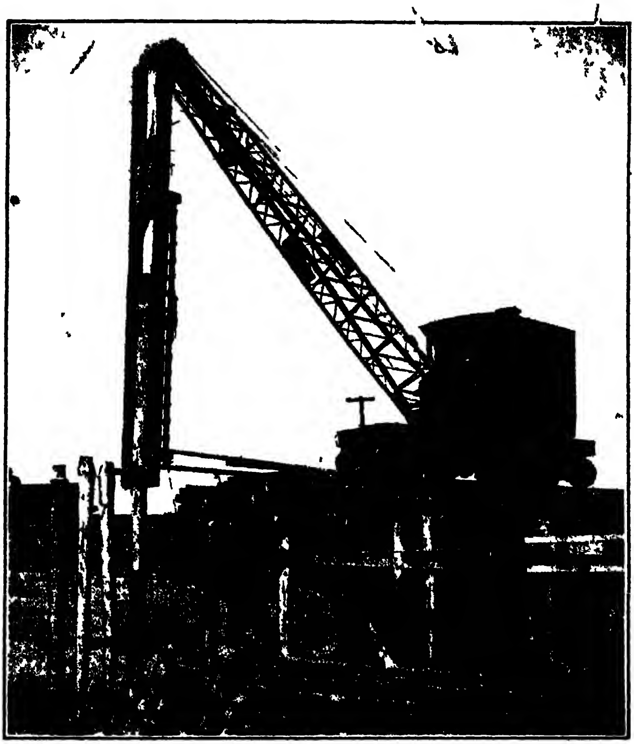


FIG. 7d—Locomotive Crane Used as a Traveler and Pile-Driver in Building a Pile Trestle.

The 25-foot leads swing freely on the bolt by which they are suspended from the boom, when driving they are braced by struts to some of the finished work Cross-pieces on the back of the leads and an iron bar placed across the front on two hooks at the bottom of the leads hold the pile in position. The drop-hammer is operated by the regular hoisting rope, and the same rope is used to hoist the pile into the leads, the hammer meantime being held at the top of the guides by a bolt

ing down temporary structures and old bridges, or clearing up a wreck. A boom is therefore provided of sufficient capacity for such work. In some instances it is placed in front of the leads when in use, while in others the boom always remains in place, being connected by blocks and tackle to a transverse frame and mast, the pile driving being done by leads hanging from the boom. In transit the boom is down and extends over the length of a flat idler car coupled ahead. In another typical form the leads and their supporting truss and braces are replaced by other appliances to convert it into a locomotive crane or excavator.

ART. 8. THE DROP PILE-HAMMER

A drop-hammer is one which is raised by means of a rope and then allowed to drop. It consists of a solid casting with jaws on each side which fit into the guides of the pile-driver leads, with a pin near the top for the attachment of the rope or of the nippers, and with a broad base on which it strikes the pile.

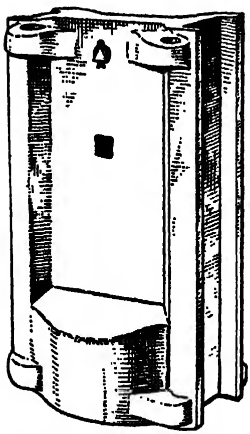


FIG. 8a.—Drop-Hammer.

Figure 8*a* shows a drop-hammer of modern design with all corners rounded. It is made as long as practicable to increase the bearing in the leads, while the jaws have as little play as possible between the leads and hammer to reduce the jar on the driver when the pile is struck. The form is arranged to have its center of gravity as low as possible. When the hammer is to hit the head of the pile directly, its base is made slightly concave, but when a pile cap is employed, as is done in the best practice, the base is made flat.

When the hammer is to have a free fall, as may be required on test piles or for very light hammers raised by horse power, the pin is triangular in section with its lower face horizontal,

to engage the "nippers" automatically. The upper ends of the nippers are curved so that when the trip is reached they are drawn together and thus release the hammer for its drop on the pile. When the hammer is to be raised by a hoisting drum with a friction clutch, a round pin is used to which the line is attached directly. The latter method affords the following advantages for regular work: more rapid operation; facility in regulating the height of drop; and avoiding the danger of losing the hammer if it should pass out of the leads.

The weight of drop-hammers most generally used in American practice to drive timber piles ranges from about 2000 to 3800 pounds. For posts and very small piles the weight runs as low as 500 pounds, while for heavy construction requiring very long piles it runs as high as 5200 pounds. For very light service a heavy block of oak wood is sometimes employed. The weight of drop-hammers to be adopted depends upon the weight of the piles and the character of the ground to be penetrated. The relation of the weight and fall of the hammer to the bearing power of piles and to success in securing adequate total penetration without injury to timber piles is discussed in Art. 29. The weight of hammers to drive concrete piles is referred to in Art. 50.

ART. 9. THE STEAM PILE-HAMMER

A steam-hammer is one which is automatically raised and dropped a comparatively short distance by the action of a steam cylinder and piston supported in a frame which follows the pile. It was invented in England by JAMES NASMYTH in 1845, and was first used on Oct. 6, 1846, to drive piles for a bridge foundation. One type of steam-hammers has been built in this country since 1875 and after various improvements has continued in use, being known at present as the Warrington-Vulcan hammer. Steam-hammers are of two general classes, single-acting and double-acting. In the former and older class the steam pressure is applied to raise the striking part of the hammer, while it falls by gravity. The force of the blow depends upon the length of stroke and the movable weight, the number of

blows depending upon the steam pressure. In the latter class the steam pressure raises the hammer and also reinforces the action of gravity during its descent, the force of the blow, as well as the rapidity of action, being functions of the pressure. The latter apparatus is more compact, lighter and operated with

FIG. 9a.—Warrington-Vulcan.

FIG. 9b.—Industrial Works.

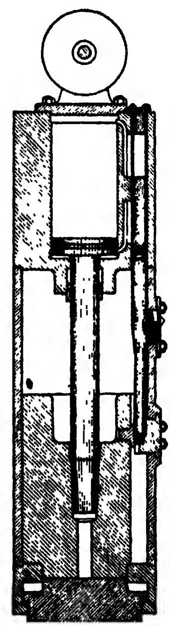


FIG. 9c.—McKiernan-Terry.

Steam Pile-Hammers.

greater rapidity. The Warrington-Vulcan is single-acting, while the Union, Industrial Works, National, and McKiernan-Terry hammers are double-acting. Another classification may be based upon whether the striking part is attached to a movable piston or to a movable cylinder. The Warrington-

Vulcan, Union, National, and McKiernan-Terry have the former arrangement, while the Industrial Works has the latter.

The following table gives weights, dimensions and other data for the largest regular size of hammer for each of five makes. It is noted that in the double-acting hammers the weight of striking parts is only about one-half to one-fourth as great as in the single-acting ones. The table also indicates the steadily increasing number of blows, as well as the reduced height, thus requiring less space in the leads. The piston speed is nearly uniform. A number of steam-hammers may also be operated by compressed air.

LARGEST SIZES OF VARIOUS STEAM PILE-HAMMERS

Trade designation	Warrington-Vulcan	Union	Industrial Works	McKiernan-Terry	National
Size number	0	00	....	11B	1
Total weight, pounds	16,250	20,000	6,600	13,185	8,000
Weight of striking part, pounds	7,500	4,000	1,900	3,625	1,500
Total height, inches	180	......	113	110	94
Diameter of cylinder, inches	16.5	14	8	12¾	9
Stroke in inches	48	36	24	20	16
Total downward force, pounds	7,500	16,320	6,175	13,200	5,300
Work done per blow, foot-pounds	30,000	48,960	9,650	22,000	7,090
Number of blows per minute	50	100	100	120	115
Boiler pressure, pounds per square inch	......	100	90	80	80
Boiler required, horse power	60	125	50	60	60

The total weights of the smallest sizes are, respectively, 1400 850, 6000, 145 and 150 pounds. The number of blows per minute for the same sizes are 80, 349, 110, 506 and 400. Generally, the lightest hammers are used for light sheet-piling only. Additional data may be found in the illustrated catalogues published by the manufacturers.

During the operation of driving, the steam-hammer and its frame rest upon the pile, the head of which is trimmed to fit into the recessed or open base of the frame. The frame has channel or angle guides on the sides which engage the leads of the driver. The frame in turn guides the hammer in its move-

ment, and in several makes entirely encases it. While the weight of the striking parts is only a fraction of the total weight, the extra dead weight of the frame helps to keep the pile in motion after it is started by the blow. Generally, the blows follow each other so rapidly that the pile is in continuous motion. The limited vibration thus developed in the pile is also an aid in securing its penetration, particularly in ground containing a large percentage of sand which otherwise offers considerable resistance. The vibration is limited by the weight which constantly rests upon the pile. The effect of the short, quick blow in securing penetration is analogous to the method of driving an ordinary pin into a block of lead by many light taps with a very small hammer, which could not be done by fewer but heavier blows.

ART. 10. ADVANTAGES OF STEAM-HAMMERS

The following selected records of actual experience are presented in order to indicate the relative values of steam- and drop-hammers when used under practically the same conditions. The interests of good practice would be materially aided if more tests of this kind were made under a wide range of conditions.

In driving piles for cylinder piers 20 feet in diameter for a bridge on the Norfolk and Western Railroad, at Norfolk, Va., the piles in one cylinder were driven by a 3300-pound drop-hammer with a fall of 10 feet, while those in the twin cylinder of the same pier 33 feet away and in the nearest cylinder of the next pier, 44 feet distant, were driven by a steam-hammer with striking parts weighing 3000 pounds, a total weight of 6000 pounds, a normal stroke of 36 inches and an effective fall of 30 inches. The following record relates only to the averages for the first six piles driven in each cylinder respectively. In the first cylinder the average penetration under the last blows was ½ inch, and for each of the other cylinders the average penetration under the last 100 blows was 7 inches, or practically 14 blows per inch. The total penetrations averaged 28, 36 and 26 feet respectively for the three cylinders. The drop-hammer

broomed the heads of the piles and no increase in penetration was secured by increasing the drop above 10 to 15 feet. With the steam-hammer no brooming occurred and the full force of the blow was effective at all stages of the driving. The piles were driven in about 38 feet of water, about 16 feet of the soft silt having been dredged out so that all the penetration secured was through firm blue mud and sand in layers of varying thickness.

On the Chicago and Eastern Illinois Railroad the performances of a steam-hammer and drop-hammer were compared by using them on the same pile, changing the hammer in the leads as quickly as possible. The former had a total weight of 5170 pounds, and striking parts of 2840 pounds, while the latter weighed 2900 pounds. The former had a drop of 28 inches, and the latter of 32 feet. With the steam-hammer, 66 blows produced 1 foot of penetration in 1 minute, and another foot by 83 blows in 1½ minutes; the next foot of penetration was obtained by the drop-hammer with 12 blows in 2 minutes, and the following 4 feet respectively by 12, 10, 10 and 12 blows in 2, 2, 3½ and 2½ minutes; the steam-hammer being replaced caused the next foot of penetration with 203 blows in 3 minutes, and the following 9 feet by 341 blows in 5 minutes.

In driving piles for a large wharf and warehouse at Pensacola, Fla., requiring 7000 piles, two piles 75 feet long were driven 3 feet apart, one by a drop-hammer and the other by a steam-hammer. The former was driven by 120 blows in 50 minutes, dropping the hammer from the top of the 75-foot leads; and the latter by 130 blows in 90 seconds. As should be expected under such abnormally high falls, the former pile was broomed for a depth of over 3 feet at the head, while the one driven with the steam-hammer was not broomed at all. The piles were creosoted and cost 40 cents per linear foot delivered.

On the North River at New York City piles from 55 to 60 feet long were driven from 43 to 50 feet below water through a 10-foot layer of cobble stones, and layers of very fine sand, coarse gravel and sand gravel, as shown by test borings. To drive 12 piles in 10 hours by a crew of 10 men was regarded as a

good day's work, an average of 175 blows with a 3300-pound drop-hammer falling 10 feet being required, at a rate of 15 blows per minute. With a crew of two or three men less, 18 piles per day could be driven by a steam-hammer and braced, some of the piles requiring over 1200 blows at the rate of 60 per minute without showing any sign of brooming. The hammer had a total weight of 8400 pounds, and a striking weight of 4000 pounds.

A contractor endeavored to drive some 45-foot piles through sand, gravel and boulders, for bridge piers on the New York, Westchester and Boston Railroad at Pelham, N. Y., using a 3000-pound drop-hammer falling 20 to 40 feet, but did not succeed. A steam-hammer was then obtained with a 3000-pound striking weight, which secured the full penetration without brooming or splitting any piles.

The following advantages are claimed for the use of the steam-hammer by those who have also had experience with the drop-hammer: (1) The pile is held in position and guided more firmly while driving, thus keeping the pile from dodging, or getting out of line, and avoiding the labor of toggling. (2) Serious damage to the pile, such as brooming, splitting, etc., is avoided. Hence piles of softer wood may be employed. (3) Extra time and cost for the use of a ring on the pile head is saved. (4) The driving is equally effective for any position of the pile head in the leads. (5) A pile may be driven several feet (7 or 8 feet with some hammers) below the bottom of the fixed leads without the use of extension leads. A few feet may often be saved in cut-off by thus driving below the elevation of rail. (6) When driving into soft material or into sand, the rapidity of action keeps the pile in motion and prevents the earth from recompacting around the pile until the driving ceases, thus reducing the frictional resistance. (7) More piles can be driven in a given time and often with a smaller crew. (8) The steam-hammer has been used effectively in places and under conditions where it was found to be impossible to use a drop-hammer successfully. This relates to cases of limited head room as well as to difficult subsurface conditions. (9) Less

injury is caused to adjacent foundations, and less breaking of glass and plastering in adjoining buildings. (10) The leads last about three or four times as long as when a drop-hammer is used. (11) On track pile-drivers less injurious strains are caused in the car and machinery, thus reducing the cost of maintenance. (12) Although the first cost of the steam-hammer is much greater, the total cost of driving is reduced.

The teaching of experience is indicated by the fact that in the city of Chicago, where perhaps more piles are driven for foundations than in any other place in the United States, steam-hammers are used almost exclusively. Those who have had considerable practice in the use of both kinds of pile-hammers do not, as a rule, wish to go back to the drop-hammer. Exceptional cases have been reported in which a steam-hammer has been unable to force a pile through a hard crust. A drop hammer may succeed in such a case because of its heavier blow, but it is more likely to break the pile. Perhaps a pointed shoe may be needed on the pile, or a charge of dynamite, or a dredge. Sometimes more caution is needed with a track driver when the track is out of level if the heavier steam-hammer is near the top of the leads.

Art. 11. Rings and Caps

It is important to cut off square the butt of a pile, so that the impact of the hammer may be distributed uniformly over the surface. Since the butt tends to change its position slightly in the leads during driving, it has been found advantageous by experience to make the lower surface of the drop-hammer slightly concave. This provision counteracts the tendency toward lateral movement of the pile to some degree. When the pressure on any fibers exceeds their ultimate resistance in compression, they will yield by bending, buckling or crushing, after their adhesion to adjacent fibers is destroyed. When the fibers are once broken down, every blow of the hammer tends to injure the fibers further down. As wooden fibers are far more compressible when a force is applied on their

sides instead of their ends, the bruised head of the pile thus becomes more elastic, and acts somewhat like a spring or cushion. When the height of fall for the hammer exceeds a certain value, a part of its energy is expended in destructive work like that just indicated, leaving less for useful work, reducing its efficiency in forcing the pile to penetrate the ground. This breaking down of the fibers is called "brooming." The fall of the hammer may be so great that nearly all of the energy is used up in brooming the pile. The relation of the weight of the hammer and the height of its fall to the bearing power of a pile is discussed in Art. 29.

It is often found that no increase in penetration is secured by increasing the fall or drop above 10 to 15 feet. It is possible to estimate approximately the loss of energy due to brooming by comparing the number of blows required per foot of penetration before and after cutting off the broomed top. From the record of a pile driven by a steam-hammer, under the direction of D. J. WHITTEMORE, it is observed that in driving the pile from the twelfth to the twenty-second foot of penetration, 4682 blows were struck, or an average of 468 blows per foot. Immediately after cutting off the broomed top at two different times, only 275 and 213 blows respectively were required to drive the pile the next foot. Their average of 244 blows indicates the number required under the condition of a sound head, and accordingly it appears that on the average only about 52 percent of the available energy was consumed in securing the penetration of the pile. The loss in this case is considered excessive. The progressive effect of brooming is shown in the number of blows required for the tenth to the fourteenth foot of penetration, respectively: 73, 109, 153, 259, 684.

The brooming and the splitting of pile heads vary for different kinds of wood. The record of pile driving for the foundations of a building in Chicago shows that for pine 12.5 percent of the heads were crushed and 5 percent broken; for gum, 7 percent crushed and 0.6 percent broken; for oak, 5 percent crushed and 0.8 percent broken; for hickory, 3 percent crushed and none

broken; and for basswood, 8 percent crushed. In several of the oak piles the sapwood and heart separated, the heart core being driven through the shell. A cast-iron cap was used in driving, but, in spite of this, an average of 8 percent of the heads were crushed or split; but when it is considered that the fall of the hammer for each pile was permitted to reach the magnitude of 35 and 40 feet when the driving ceased, it is surprising that these percentages were not larger. The penetration at the last blow averaged 3 inches. The percentages of piles used on the work of the different species of wood named above and in the same order were 22, 32, 21, 15 and 7 respectively.

The crushing of the fibers is frequently followed by the splitting of the pile head. This tendency is promoted by failing to cut off enough of the butt as it comes from the forest to cover the entire section area of the pile, for if the hammer hits only one-half of the area it will force that part down into the head and split it.

To prevent splitting and to reduce brooming, the head may be hooped by a pile ring. The sizes range from 2 by 3/8 to 4 by 1 inches. The diameters vary to suit different sizes of pile. They are made of the best quality of wrought iron that can be obtained. Rings of the best bar iron usually last to drive 50 oak piles or 200 cedar piles; those of the best hammered iron for 75 oak piles or 300 cedar piles Rings made out of old car axles have been used for 250 oak or 6000 cedar piles.

In fitting the ring the pile is neatly chamfered down at least 5 inches from the end, so that the ring will just catch on; a blow of the hammer puts it into place. To remove the ring, a cant-hook or pevee is used, the pile line being fastened to its end to apply steam power. If the pile brooms too much in spite of the ring, the recognized remedy is to saw off the broomed part, so as to present a solid surface to the hammer and put the ring on again.

For small jobs the use of a spiral coil of No. 9 wire on the top of the pile has proved satisfactory in preventing brooming. The coils are made on a conical block, from six to eight turns being used. When the pile is ready for driving, the coil is

placed on top, flattened down and secured with staples. The first blow of the hammer sinks the coil into the wood, thus affording a spiral reinforcement.

A more effective and less expensive method of protecting the head of a timber pile from brooming and splitting is the use of a pile cap as shown in Fig. 11*a*. It consists of a casting with a tapered recess above and below. The chamfered head of the pile fits into the lower recess and a short cushion block of hard,

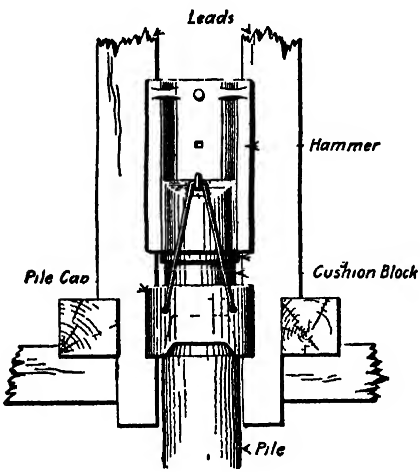


FIG. 11*a* —Casgrain's Pile Cap

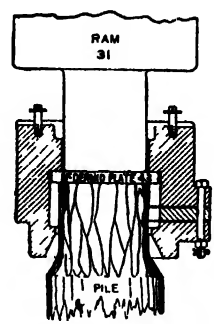


FIG. 11*b* — McDermid Base.

tough wood is fitted into the upper one. The block is frequently provided with an iron hoop or ring around its top. The cap has jaws on the sides like the hammer which engage the leads, and hence the head of the pile is held in position and guided while driving. After the pile is driven, the cap is hooked to the hammer by ropes and pins and raised with it. While the cap protects the pile head, the short cushion block requires frequent renewal, since it gets the direct impact of the hammer. Sometimes a rope mat is placed on top to protect it. White, live or swamp oak, rock maple and blue gum have given good service for cushion blocks.

When both drop- and steam-hammers are used on the same work it is often found that the drop-hammer causes brooming

when the steam-hammer gives no indication of it. In hard driving, however, it becomes important to protect the pile head. Sometimes this is done by spiking a flat steel plate on the pile to receive the blow, or a dished or cupped striking plate may be substituted for the flat plate. A better arrangement is adopted for some makes of steam-hammers. The Warrington hammer substitutes for its ordinary base what is known as the McDermid patent base (Fig. 11*b*), in which a recess is provided for a thick steel plate inserted through a slot in the side, covered by a door. The plate is held in place by the base and thus avoids the danger to the crew which occurs with the separate flat or dished plate. The Union, National and McKiernan-Terry steam-hammers are provided with an anvil block, in the base, which rests on the pile.

ART. 12. FOLLOWERS

When a pile has to be driven below the leads, or below the ground or water surface, a follower is generally employed. A follower is a member interposed between the hammer and a pile to transmit blows to the latter when below the foot of the leads. In its simplest form a follower may consist of a short pile or stick of white oak of the requisite length and diameter. To keep its lower end in position on the pile, a follower band may be used which is flared both upward and downward, but it is better to use a follower cap. This is a cylindrical casting with a horizontal diaphragm at the middle, which is bolted to the lower end of the timber follower, and fits over the head of the pile. The upper end of the follower is held in position by the recessed base of the steam-hammer or by a pile cap if a drop-hammer is in use.

A better kind of follower consists of an extra strong pipe cast into the follower base, so as to avoid the objections to the use of bolts. A stick of turned hardwood is driven into the pipe. An iron band is shrunk on the pipe so as to project beyond the top into which is fitted a hooped oak driving block that may be replaced when worn out. Patented followers are also used, to which pipes are attached by which steam or air may be intro-

duced on top of the pile to release the follower when such aid is needed in certain soils. When followers are used to drive piles through a considerable depth of water, the base of the follower should engage extension leads so as to hold and guide the head of the pile properly. In deep water with a swift current it may not be possible to handle the follower effectively. In such cases long piles are driven while their heads remain above the surface; afterward they are cut off at the proper elevation.

In placing piles for a bridge over the Maumee River in Toledo the piles, 19 to 25 feet in length, were placed in 35 feet of water by first lowering a 19-inch steel tube 45 feet long to the blue clay, through which piles were driven by means of a follower. The bottom of the tube extended a few inches into the clay while the top was held in the leads. The 46-foot timber follower was just large enough to fit neatly inside the tube, and it had a steel sleeve at the bottom which encased 18 inches of the top of the pile and held it true.

A follower generally absorbs a considerable percentage of the energy of the hammer, frequently amounting to 50 percent. The loss is greater when the lower end of the follower is not guided by the leads and the pile is set into unusual vibration. The following record by J. E. CRAWFORD shows, however, that under proper conditions there may be no appreciable loss in the effect of the blow. The pile sank of its own weight 6 feet, then the hammer with its housing weighing 6000 pounds was put on it, and it sank 5 feet further. The number of blows for each succeeding foot of penetration were 9, 5, 13, 20, 14, 16, 17, 15, 30, 40, 47, 65, 45, 26, 22, 33, 60, 55 and 55. Then the follower was put on and the number of blows required per foot were 55, 75, 56, 60, 73, 90, 113, 115 and 102 blows for the last 7 inches, giving the pile a penetration of 39 feet 7 inches.

The type of hammer shown in Fig. 9*c* drives piles below the water surface without the aid of a follower. To keep water out of the lower cylinder, compressed air is supplied through an air hose connected to the manhole cover. The exhaust from the hammer is carried to the surface through a hose.

The hammer slides on a carriage consisting of a steel channel, which in turn, slides on a 12- by 12-inch timber spud. A T-rail is fastened to the spud and Z-bars are riveted to the web of the channel to engage the head of the rail. A second set of Z-bars are fastened to the hammer to engage the flanges of the channel.

The hammer is raised in the carriage and the carriage then raised to the top of the spud, after which the whole is raised as a unit. The pile is then placed in position, the top inserted in the bell-bottom anvil block of the hammer and the lower part fastened to the bottom of the carriage. The whole is then lowered until the spud rests on the bottom, the latter then being brought to a vertical position, after which the carriage and hammer are lowered until the pile touches bottom, and driving is then commenced. Piles have been driven by this method with a hammer submergence of 70 feet.

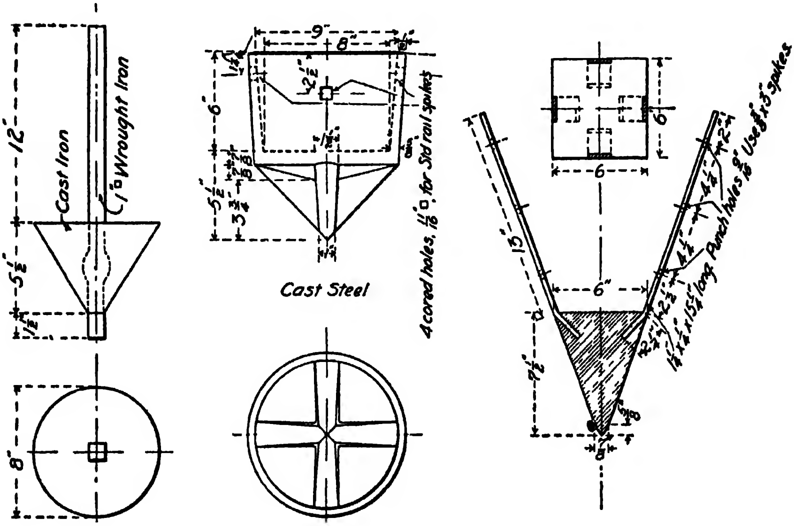
Art. 13. Points, Shoes and Splices

The foot of a timber pile should always be cut off perpendicular to its axis, since it facilitates driving it true to line or position. In soft and silty ground, or where the driving is easy, it is not necessary to sharpen or point the pile. If a pile penetrates soft material and rests upon a hard stratum, thus acting as a column, the unpointed foot has the additional advantage of providing a larger bearing area. The blunt end on striking a root or any small obstruction will generally break the obstruction without deflecting the pile.

In driving a pile with a blunt end, a cone of compressed earth forms under it and acts in most respects as if the pile were pointed. It is frequently claimed that even in driving through hard material a pile will keep more nearly to the required position than if it is pointed. This implies that the cone of earth is more likely to have the form of a fairly good cone or pyramid than the wooden point made by sharpening the pile. Such a contention can hardly be maintained if the pointing is properly done. When coarse gravel or boulders are encountered which destroy the cone of compact earth, crush the

fibers of the timber and wedge them apart, it is desirable to reduce the area of the foot by pointing. In general, when the ground is at least moderately compressible and the driving is not hard, the foot of the pile may be left unpointed.

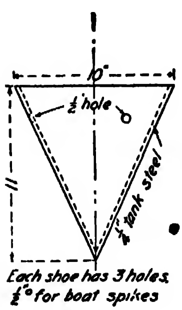
When the driving is hard for most of the penetration, as in stiff clay or in material that is but slightly compressible, and hence must be displaced, it is advisable to point the pile, so that it may separate the material at the foot like a wedge. In pointing a pile it is preferably sharpened to the form of a trun-

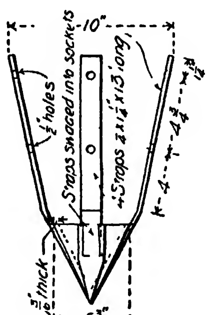


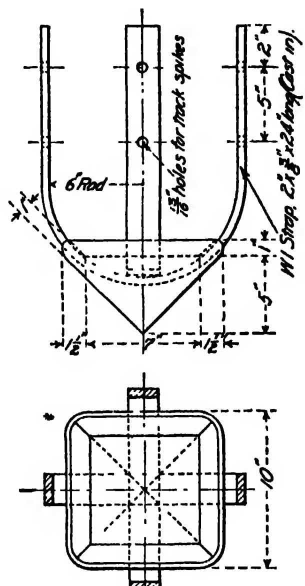
FIGS. 13*a*, *b* and *c*.—Shoes for Timber Piles.

cated pyramid, the end being from 4 to 6 inches square. If the end is too small the fibers lack the necessary strength to resist brooming. The length of the point may be from one and a half to two times the diameter of the foot. Another advantage of pointing is to increase the rate of penetration, or to reduce the energy required. In compact material the bearing power of a pile is practically the same with or without the point. Experience has also shown that piles with pointed ends may be successfully driven through old timber cribwork while attempting to drive them with blunt ends resulted in broomed tips, split and broomed heads.

Sometimes the timber point is replaced or protected by a metal shoe. Figure 13*a* shows an undesirable form which tends to split the pile when the side of the shoe strikes an obstruction. Figures 13*b*, *c* and *f* illustrate the best forms, since the timber has a square bearing on the upper flat surface of the shoe and the sides of the socket or the straps permit such a firm fastening as to make the shoe act like an integral part of the pile. Those in Figs. 13*d* and *e* are not quite so effective unless a close fit is secured in the socket at an increased labor cost. Shoes are used by some engineers when piles are driven into material containing boulders, riprap,







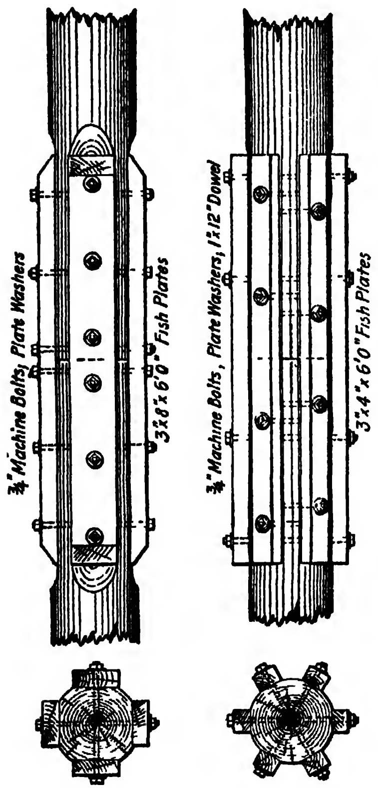
FIGS. 13*d*, *e* and *f*.—Shoes for Timber Piles.

coarse gravel, shale, slate, hardpan, buried timber, very hard clay and coral rock. Another use is to penetrate a thin, hard stratum (2 feet or less) which overlies a softer one. They are also attached to piles for bridge falsework in order to gain a foothold on rock bottom. In one case it was thus possible to secure sufficient penetration to hold the piles against a 20-foot rise in the river and a swift current.

On the Key West Extension of the Florida East Coast Railway, where numerous pile foundations are built on coral rock containing pockets of different sizes, a hole was made by driving a steel punch with the pile-hammer and then driving in the

timber pile with a few light blows. In order to permit the punch to be withdrawn readily, it was provided with a foot slightly larger in diameter than its body.

Some engineers and contractors condemn the use of shoes unqualifiedly because of their unsatisfactory experience, but in many cases such experience is probably due to employing shoes which were improperly designed or constructed, while in others the piles should have been omitted, since the ground was hard enough to support the substructure directly.



FIGS. 13g and h.—Fish-Plate Splices for Timber Piles

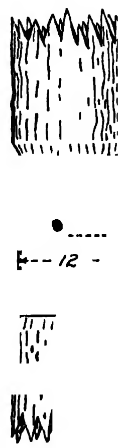


FIG. 13i.—Tubular Splice for Timber Pile.

It is occasionally necessary to use longer piles than can be obtained in single sticks. It becomes necessary, therefore, to splice two piles together end to end. For this purpose a fish-plate joint is usually the best, since it provides lateral resistance. Either four or six timber fish plates may be used, as illustrated in Figs. 13g and h. For the falsework to erect

the Poughkeepsie bridge, 55-foot piles were spliced to 75-foot piles by means of fish plates 20 feet long, eight fish plates 4 by 5 inches in section being fastened to the piles with ½-inch wrought-iron spikes 8 inches long. The water was 55 feet deep. Wrought-iron fish plates may be employed instead of wooden ones and thus reduce the sectional area at the joint. Another method is to use a metal sleeve consisting of a piece of heavy pipe as indicated in Fig. 13*i*. Half-lap joints fastened either with bolts, bands or wire wrapping are often used, but they are deficient in lateral strength and stiffness.

In rebuilding the fender piers of the Thames River bridge, in 1902, 400 piles were driven, formed by splicing spruce piles 35 to 40 feet long to creosoted yellow pine piles from 50 to 65 feet long. The water was 50 feet or less in depth, so that the spruce piles are below the bottom of the river and hence free from the attacks of the teredo.

Pile splices may also be required where piles have to be driven in sections on account of limited clearance under a bridge. Piles in three sections have thus been placed with pile-drivers having short leads. The sections were joined together with iron sleeves, the piles being found satisfactory under test loads. In swampy places one pile is sometimes driven on top of another with only a dowel connecting the two. Such a joint affords practically no lateral stiffness and the upper section is liable to bounce off while driving unless the dowel is very long.

In pile trestles where the upper portions of long piles are decayed, repairs may be made by cutting out the decayed section and inserting new timbers. In one case four steel angles were used as fish plates for each pile. They were well fastened with spikes, and each end of the joint was wrapped with a band of heavy wire, while spiral wrapping extended between them. A shell of concrete was then cast around the joint to protect the metal. The repairs cost about 15 percent of the cost of the piles in place.

CHAPTER II

DRIVING TIMBER PILES

ART. 14. OBSERVATIONS IN PRACTICE

As a general rule, a heavy hammer with a low fall secures greater penetration with less expenditure of power than a light one with a high fall; it is also less injurious to the equipment. More blows can be given in the same time with a low fall and hence less time is given between blows for the ground to compact itself around the pile. In quicksand it is especially necessary to have the blows follow each other as rapidly as the operation of the hammer permits. In silt the rapidity of blows need not be quite so great as for quicksand.

When a pile sinks at a uniform rate it is less apt to jam, buckle or split than when driven with heavier blows and with marked intervals of time between them. This statement is confirmed by observations in putting down steel sounding rods by hand. For example, through soft gravel mixed with quicksand, one man may be able to push a rod down 5 or 6 feet, and if quick enough may pull the rod up again with the same expenditure of energy. If, however, the rod is allowed to rest no longer than 15 seconds, the sand packs against it so that two men are scarcely able to pull it up. A pile which is left standing for a few minutes in some kinds of sand may be packed so hard as to resist further penetration, or at least to require a much larger impact to start it again.

A very slight bounce of a drop-hammer occurs at every blow under good conditions for driving, but decided bouncing of the hammer may occur when the penetration ceases, or when the hammer is too light, or the fall too great, or both; or when the head of the pile is crushed or broomed so as to cushion the blow.

In certain kinds of soil a pile may sink some distance and then refuse to go further, but will resume penetration when driven after an interval of rest; or it may refuse to sink under a heavy hammer and yield under the more rapid blows of a lighter one. The driving of one pile may cause adjacent piles to rise, and in soft ground or mud often causes an adjacent pile previously driven to move away slightly.

The great variety of experiences which may occur on a simple work of construction may be illustrated by those encountered in driving piles for the Ogden-Lucien Cut-off of the Central Pacific Railway. The nature of the bottom of Great Salt Lake was found to be so variable that at times a blow of the hammer drove a pile only 1 or 2 inches, and at other times 1 or 2 feet; or a pile seemed to strike a hard stratum and refused to sink farther under many blows, but after being forced through, the pile sank as much as 2 or 3 feet per blow. Frequently, a pile with a penetration of 30 to 50 feet would suddenly rise 2 or 3 feet during a short delay of the hammer. At the end of the temporary trestle, to be later replaced by a rock fill, a new difficulty was encountered. The first pile 26 feet long was driven out of sight by a single blow, and when another pile 28 feet long was placed on top of it, the next blow of the hammer sent both out of sight. The formation was found to be a deep mud deposit due to the Bear River. As the mud was 50 feet deep, two 40-foot piles were driven on top of each other. The trestle supported by these spliced piles supported the trains until the rock fill was completed and settled.

In certain kinds of clay the lateral spring of a pile under the hammer blows makes a hole slightly larger than the diameter of the pile, allowing surface water to find its way to the foot of the pile thus reducing both the skin friction and the bearing power of the clay under the foot of the pile. This action explains why cases have been observed where piles settled under moving trains after a rain although the resistance of the pile when driven was considered satisfactory. The treatment of such conditions is indicated in Art. 17.

The principle involved may be advantageously applied in some cases to reduce the resistance in pile driving when there is no available water-jet equipment. For example, by discharging water on the surface of the ground at the pile with an ordinary garden hose, without a nozzle, the number of blows by a steam-hammer was reduced from 296 to 164 for a 45-foot pile in a Chicago building foundation.

One engineer declared in a discussion on this subject that sometimes hardly a day passed but someone rushed into the office to state that in a certain place the piles were being driven too deep; that they had gone through a hard stratum into a weaker one, forgetting that in ground where its supporting power depends mainly upon skin friction the total penetration must be large.

A frequent cause of small penetrations per blow is the crookedness of a pile which produces a lateral spring under the hammer blow, and thus dissipates some of the energy. Occasionally, it is due to the head of the pile being cut off improperly, so that the hammer strikes on one side only. Perhaps the most common cause is due to setting the pile out of plumb in the leads, on account of undue haste or carelessness. It is equally as important to keep the leads plumb by leveling up the tracks on which the pile-driver moves

Underground conditions sometimes force a pile out of line in spite of ordinary efforts to control its movements in the leads. A block and tackle or a jackscrew may be required to force it back.

If it is desired to compact the ground in a given area uniformly, it is best to begin driving piles at the center and work outward to the perimeter. If the order of procedure is reversed, it becomes more and more difficult to drive the piles toward the middle to secure the same penetration and usually the adjacent outer piles will be forced to rise more or less.

It is often instructive to notice the effect on piles in places when additional ones are driven near-by. One may observe piles which were cut off to grade rising from 2 to 3 inches when adjacent piles are driven, showing that the ground between

the piles is thoroughly compacted and that its vertical motion indicates the line of least resistance. If time permits, they may be given some extra blows to settle them, or they may be cut off again to grade, since the phenomenon shows that the full supporting power which the nature of the ground permits is being applied to the pile.

On foundation work for the Illinois Central Passenger Station at Chicago a group of eight piles had been driven, sawed off to a uniform height, and wales drift-bolted to them. Upon driving a group of 16 piles 15 feet away, the piles in the former group rose 4 inches next to the driver and 1 inch on the opposite side. In a group of 72 piles, observations were taken daily on the head of the first pile while the rest were being driven. The pile sank ½ inch during the first two days, then rose steadily until 50 piles were in place, when it was 3 inches above the original elevation, the greatest rise in one day being ¾ inch. The pile was 55 feet long, and had a total penetration of 45 feet.

The distance to which vibration was felt at the same site varied with the height of fall of the hammer, the nature of the ground and the spacing of the piles. The vibration was easily felt at a distance of 400 feet and was quite marked at 75 feet. In doing instrumental work it was sometimes observed that the vibration within 25 feet of the pile-driver was less severe than at several times that distance.

Art. 15. Driving Piles Butt Down

It is the general practice to drive piles with the tip downward. Occasionally, however, special conditions make it advisable to drive them with the butt downward. It has been found difficult at times to keep a pile down after being struck by the hammer, the pile beginning at once to rise, lifting the hammer with it; and upon raising the hammer the pile may shoot upward 5 feet or more, or the pile may exhibit this tendency but slightly when driven, but the following morning will stand with its head a number of feet higher than before. This behavior

is ascribed to a substratum of quicksand, and the difficulty is usually overcome by driving the pile "butt down."

Another condition occurs when piles are driven through very soft ground and the load has nearly all to be borne by the foot. The substratum may require the larger bearing area afforded by the butt of the pile to carry the load. Some engineers recommend that tall pile trestles which are to be filled should have the piles driven butt down, thus leaving no hollows to cause trouble as the embankment settles. The bracing can be removed as the filling rises. In hard material the butt may have to be pointed to a smaller diameter to facilitate penetration. Great care must be exercised in driving on account of the smaller area of the tip, which receives the blow, the smaller percentage of heartwood in that area and the weaker fibers of the wood which grows in the upper part of a tree trunk.

In some cofferdam construction on the Ohio River where it was necessary to drive about 600 oak guide piles into hard gravel it was found that the best way to secure adequate penetration was to drive them with the butts down. In this manner the resistance encountered due to the wedge action of piles as usually driven was avoided, and the useful effect of the blow was all transmitted to the foot of the pile.

An interesting example relates to piles driven 4 to 6 feet apart both ways in the embankment of the Yazoo Canal near Vicksburg, Miss., to stop the bank from sliding on the adjacent railroad track during the low-water stage of the river. By driving the piles butt down, advantage was taken of the larger cross-section at the lower elevation where the bending moment was a maximum. Pile trestles have resisted the pressure of ice going out by having extra flexural strength due to the piles being driven butt down. When piles have to be driven into sand with their butts down, the water-jet should be employed (Art. 17).

ART. 16. DRIVING BATTER PILES

A batter pile is a pile driven at an inclination to resist forces which are not vertical. It is sometimes called a spur pile.

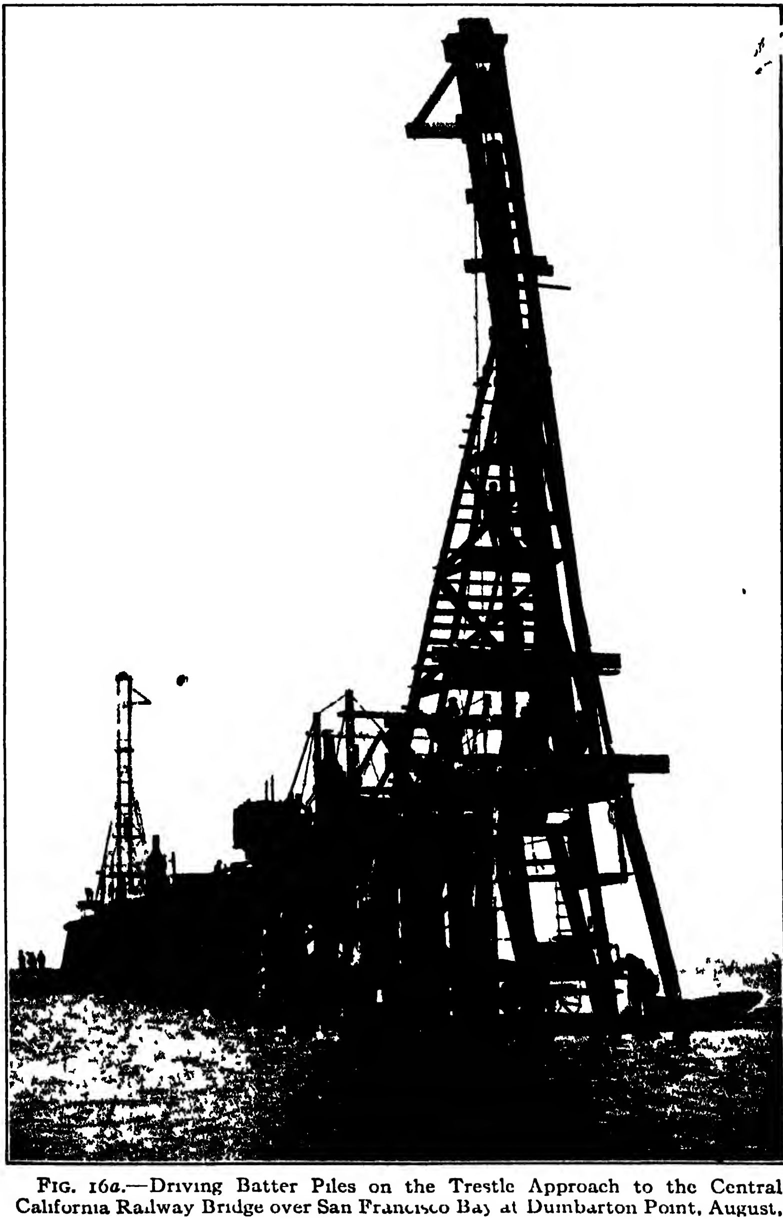


FIG. 16a.—Driving Batter Piles on the Trestle Approach to the Central California Railway Bridge over San Francisco Bay at Dumbarton Point, August, 14, 1907.

(Facing p 42)

Fig. 19a.—Examples of Overdriven Piles Exposed by Subsequent Excavation

When a pile-driver is designed to drive batter piles as well as the ordinary vertical piles, its leads are suspended from a horizontal pin to permit them to be swung laterally like a pendulum. Hence they are known as swinging leads and sometimes as pendulum leads. The pivot is attached to the top of a tower, the front timbers of which are inclined laterally to provide the requisite transverse bracing. Figure 16*a* shows batter piles being driven in the trestle approach of the Dumbarton bridge across San Francisco Bay. Although this illustration is that of a track driver, the arrangement of swinging leads shown is the same as for ordinary land drivers or for floating pile-drivers.

Occasionally, drivers are arranged to drive batter piles by having a removable section at the bottom of the back stays, so that the tower revolves backward about hinges located near the foot of the leads. Another scheme consists in taking a separate set of leads and temporarily bracing them to the tower of a pile-driver. In this manner, batter piles 60 to 70 feet long were driven for car-dump foundations at the Erie Railroad dock at Cleveland, the piles sloping downward toward the driver.

An interesting form of pile-driver was used in 1904 to drive piles for a permanent extension on the Ogden-Lucien Cut-off, which crosses the western arm of Great Salt Lake. It was designed to operate from a low falsework built alongside, the tower overhanging the track. The leads were arranged to swing forward below to drive the piles and whenever a train came along they could be swung back between the timbers in the tower corresponding to fixed leads. The king pin supporting the leads was placed directly over the center of the track, so that the leads had their correct position and direction to drive the respective piles of each bent when the proper pins were inserted in the struts holding the leads. Two sets of rollers were set under the sills of the driver, the lower set to allow the driver on the falsework to move parallel to the trestles and the upper to move at right angles to it, to place the driver in the clear of passing trains.

Batter piles are used under arch abutments to resist the horizontal component of the reaction, and sometimes several

are employed under each side of piers for simple truss or girder spans when the weight of the pier is not sufficient to provide adequately for the effect of traction. Quay walls are provided either with batter piles or with rods to anchor piles, or both. Many accidents to such structures have occurred because of a failure to provide batter piles to relieve the vertical piles from flexural stresses. Vertical piles in permanent structures should be protected against the action of lateral forces whenever possible either by sway bracing or by batter piles. (See Fig. 138c for an illustration of the use of batter piles in the foundation of a bridge pier.)

ART. 17. USE OF THE WATER-JET

A method of placing a pile in position, which differs radically from that of driving it with a hammer, consists in displacing the material by means of one or more jets which discharge water under pressure at or near the foot of the pile. As the water comes up around the pile carrying with it some of the material, it also diminishes the frictional resistance of the pile. In some kinds of earth the hammer is merely placed on top of the pile to increase the pressure by its weight, while in other cases the hammer is operated with a restricted fall to secure a greater rate of penetration.

In soft ground one jet may answer the purpose, but in most cases two jets, used on opposite sides of the pile, give better results. As the pile tends to move toward the side where the jet is operated, the use of two jets usually enables a pile to be placed more accurately in position. Sometimes a third jet is employed, discharging at a higher elevation than the others, if difficulty is experienced in keeping the ground from packing against the sides of the pile.

The water has a puddling action upon the adjacent earth and after the jet is removed the earth packs closely around the surface of the pile, thus securing a greater skin friction than if the pile is driven by means of the hammer alone. In order to secure better bearing at the foot, it is customary

to shut off the water just before the pile reaches its intended total penetration, and to complete the driving by a few blows of the hammer. This procedure presses the pile firmly into the softened earth and tends to avoid any arching action of the earth that might prevent the material from filling every cavity when it settles into place.

The water-jet may be used advantageously in any material that will settle around the pile after the flow of water ceases. The best results are obtained in pure ocean or river sand. In this material the simplest form of jet may be used, only a moderate pressure is required, a single jet will generally answer, the time of sinking is very short, and the sand packs quickly after the water is shut off, while no blows of the hammer are needed except for the purpose stated in the preceding paragraph. It is fortunate that this is the case, for pure sand offers very high resistance to a pile when driven with the hammer alone, especially with a drop-hammer. Even in quicksand, this is frequently found to be true. With the jet a pile may be sunk in sand without danger of injury, while it is difficult to avoid injuring piles when driven into sand without the aid of the jet; more time and a larger expenditure of energy are also required.

Piles have been driven with the aid of the water-jet process in mixtures of sand and silt or gravel, if the latter is not too coarse; in loam, clay, marl and even "gumbo" in pockets, although a special nozzle is required for the material named last. Some of the most experienced engineers in the use of the jet have driven piles with its aid in "sand and clay and the hardest kind of bottom," and in "almost any material except hardpan and rock." In hard ground, the jet process may be used advantageously in case sufficient volume and pressure of water be provided. In clay it may be economical to bore several holes in the earth before driving the pile, thus securing the accurate location of the pile and its lubrication while being driven. Where the material is of such a porous character that the water from the jets may be dissipated and fail to come up in the immediate vicinity of the pile, the utility of the jet process

is uncertain except for a part of the penetration. In mixtures with gravel or coarse material the water will often wash out the sand and finer material leaving the stones in the hole to interfere with the penetration of the pile. This action may often be remedied, however, by increasing the volume and pressure of the water.

In driving in sand the jet should be hung on a rope passing over a pulley in the driver so that it may be kept moving up and down with its point near the point of the pile. If this is not done the pipe is likely to "freeze" fast and cannot be moved. After the pile reaches a depth of 10 or 15 feet the water will sometimes fail to come up around it, breaking out on the surface at a considerable distance, perhaps around a pile driven previously. When this occurs it indicates that the jet has not been kept moving sufficiently, or an auxiliary jet discharging at some intermediate depth may be needed. In any case the jet should be withdrawn at once and immediately put down again, thus usually reestablishing the flow of water along the pile. Where piles are sunk 20 feet or more into sand, it is advisable to have two jets. One is to be kept moving with its nozzle slightly ahead of the pile, while the other is slowly raised and lowered between the foot of the pile and the surface to maintain the flow along the pile. On the other hand, when the material is soft and readily compressible as in silt, or in fine sand mixed with silt or a small percentage of clay, it may not be economical to use a jet, since the pile may be driven quickly without risk of injury by means of a steam-hammer.

The effectiveness of the water-jet is demonstrated at times during the operation of sinking a pile when a breakdown occurs, and an attempt is made to drive temporarily without its aid. So frequently will the piles broom or split, or give other signs of injury before reaching the full depth of penetration, as to preclude further driving. When piles were first driven at Atlantic City, prior to 1890, a contractor became bankrupt by attempting to drive piles in the wet sand by means of the hammer alone. A score of years later when the water-jet was generally used in that locality, contractors were wont to consider

it as an ordinary performance to sink 100 to 120 piles in a half day. This fact illustrates the saving in time and money which is made possible by the aid of the jet.

In Florida, palmetto piles are sometimes used, since they are comparatively free from the ravages of the teredo. This wood has a hard shell and a soft interior, and cannot stand heavy blows with a hammer. Such piles may be easily sunk into hard sand by a water-jet, the hammer resting on top and occasionally tapping the pile, the fall being only 3 to 6 inches. By keeping the pile and jet pipes constantly moving, the sand is kept from closing in on the pile until it occupies its final position. With the aid of the jet, piles may be sunk as readily with the butt down as with the tip down.

In general, the water-jet should not be attached to the pile, but handled separately. The nozzle is usually extended a small distance, not exceeding a foot, below the foot of the pile, but sometimes it is necessary to move it up and down to reduce the frictional resistance on the pipe, or to change its position if a boulder is encountered, so as to excavate an opening into which the boulder may be pushed by the pile. If this is not sufficient to displace an obstruction, the pile may be raised a little and dropped with the hammer resting upon it. It is not desirable to bend the pipe, as is sometimes done just above the nozzle. For depths not exceeding 15 to 20 feet and when the ground does not consist of layers differing materially in character, the average rate of penetration is often found to be remarkably uniform, independently of the depth.

At the Brooklyn anchorage of the Manhattan bridge 2500 piles were driven, about 40 feet long and 14 to 16 inches in diameter at the butt. Great difficulty was experienced in driving them on account of the numerous large and small boulders encountered and a thin stratum of hardpan that had to be penetrated. With the hammer alone test piles could be driven only 8 to 10 feet, but with the aid of a powerful hydraulic jet they could be driven to a depth of 40 feet. When a boulder was encountered the jet was worked around its edges until it was moved aside, or until it was undermined and finally sunk

to a position below the foot of the pile at its desired elevation. The continued use of the jet softened the ground when it could not excavate it, so that the pile could be driven further to its final grade. In this manner piles were driven where it would have been impossible to do so without the jet. Boulders 2 cubic yards in volume were sometimes displaced.

In using the water-jet, the quantity of water should be ample. In most cases volume rather than velocity is necessary. The velocity must be sufficient to excavate the sand below the foot of the pile and to make it "live" or "quick," while the volume is large enough to force the water to escape by rising along the sides of the pile to bring the material to the surface, and at the same time to reduce the surface friction, if it does not entirely eliminate it. In beach sand, piles have been jetted down within 18 inches of adjacent piles without disturbing them, showing that in this material the movement of the water is confined to a small radius horizontally. In cities the water-jet cannot be used as freely as elsewhere on account of the danger of settlement to adjacent foundations and injury to the heavy structures supported by them.

Where piles are to be driven to the uneven surface of an underlying ledge of rock, the proper length of pile may be determined conveniently by running the jet down to the rock and measuring the penetrating length of pipe. The bearing power of piles sunk by the water-jet process is determined by test blows of the hammer after the material has had time to settle or pack around the pile. In pure sand the penetration per blow is so small that the bearing power of the pile is limited either by the safe compressive strength of the wood of which it is composed, or by its strength as a column in case the total penetration is only a part of its length.

The earliest authenticated use of the water-jet in sinking piles appears to have been introduced on the construction of a wharf at Decrow's Point, Matagorda Bay, Tex., in 1852, and to have arisen from a suggestion made by Lieut. GEORGE B. McCLELLAN, Corps of Engineers, U S. A. The water was pumped by an ordinary hand pump through a rubber hose with

a gas-pipe nozzle, the nozzle being placed close to the tip of the pile. The historical development of the water-jet process is described at length in an article on "The Water-Jet as an Aid to Engineering Construction," by L. Y. Schermerhorn, in Proceedings of the Engineer's Club of Philadelphia, vol. 17, 1900. The use of the water-jet in driving concrete piles is treated in Art. 51.

Art. 18. Equipment for Water-Jet Process

The water-jet consists generally of a straight pipe with a nozzle at its end, connected by some length of flexible hose to the discharge pipe from the pump which provides the water under pressure. The suction pipe connects the pump with the source of water supply. The pump is operated by steam, being connected either with the boiler for the pile-driver or with a separate steam supply. Sometimes a short piece of curved pipe is coupled between the straight jet pipe and the hose. The pipe can be raised and lowered by a line attached to the top leading over a snatch block to be operated by hand power on the ground, or to a spool on the hoisting engine.

The diameter of the jet pipe is either 2 or 2½ inches. The discharge pipe of the pump is in most cases 4 inches in diameter, while the diameter of the suction pipe is 6 inches. To increase the velocity of the water and thus increase its power to loosen the earth, the size of the pipe is drawn down at the end to form a nozzle. The nozzle is usually circular in section and its diameter varies from ¾ to ½ inch. In a few cases a rose-jet has been employed, the nozzle having one central opening at the end and five openings around the sides with their axes inclined about 45 degrees to that of the axis of the pipe (see Fig. 94*b*). Another form of nozzle is made by flattening the end of the pipe until the opening is reduced to ¼ inch. This nozzle has given better results than a round one, especially in stiff material, it being rotated back and forth about its axis.

The quantity of water to be discharged varies from 50 to 250 gallons per minute. It must be sufficient to bring to the

surface the material which is next to the pile. The pressure ranges from 65 to 200 pounds per square inch, although the hose and fittings are sometimes designed to resist a pressure of 250 pounds per square inch. The higher pressures are required especially when gravel and boulders are encountered. Reciprocating pumps are employed, either single-acting or double-acting, and sometimes compounds. The error most frequently made is to use pumps of insufficient capacity, leading to ineffective work with the jet, and loss of time. Unsatisfactory results with the water-jet, due to inadequate and inefficient equipment, explain why the water-jet process has not come into more extensive use in pile-driving practice.

A device has been invented by which the jet pipe is handled by means of the water pressure, thus reducing hand labor to a minimum. By turning a valve, the operator who guides the pipe near the pile can control the direction and pressure of the water so as to raise or lower the jet or to hold it in any given position. The jet pipe acts as a plunger inside of a larger pipe which acts as a cylinder, the latter being suspended from a derrick pile-driver.

ART. 19. OVERDRIVING PILES

Examples of piles which were injured by overdriving are occasionally exposed by subsequent excavation. In Jersey City some piles were driven into the surface of a street to support temporarily a large water pipe, and afterward the street had to be excavated for the passage of a railroad. It was thought that the piles were well driven and in good condition. It was discovered that about half of them were ruined in driving, some being broken off square across and the upper piece driven alongside the lower piece. One pile encountered a large flat rock about 16 feet below the surface, and the fibers of the tip were found to have turned aside horizontally to a distance of 15 feet.

In Brooklyn, before making certain subway excavations, timber piles had been driven on the sides of the street to sup-

port temporary plank roadways. When they were exposed by the steam shovel many of them were broken, splintered or sheared. It was known from their behavior during driving that some of the piles were overdriven, but the injuries proved to be more numerous than had been supposed. The piles were mostly spruce, as yellow pine would not stand the hard driving. The average diameter at the butt was 12 inches and after a preliminary test 20 feet was found to be the most satisfactory length. A 2000-pound drop-hammer was used with a fall ordinarily of 25 feet. Although a water-jet aided in driving, it was difficult to secure the desired total penetration. The pile fractures had a variety of forms; some were splintered and broomed, others burst and were spread out, while others were sheared apart, and the upper end driven past the stub.

In western Massachusetts, where a new railroad passed under an old railroad embankment, temporary bents of piles were driven about 22 feet deep through fine, compact sand. The driving was hard and no water-jet was employed. The excavation of the bank showed over half of the piles to be seriously damaged, being split or broken at distances exceeding 8 feet below the surface. In most cases the break was a double shear, the upper part acting as a wedge to split the lower piece; some failed by a single inclined shear, and a few by bulging. Sometimes bulging assumes the condition of collapsing like an accordion.

In 1907, while sinking a pneumatic caisson for the Vancouver bridge over the Columbia River, some piles were removed which had been driven in 1890. Their tips were found to be broomed and shattered by driving into the compact gravel which they did not penetrate.

During excavation for a building in New Orleans it was seen that not all the test piles previously driven had pierced the sand stratum. Those with knotty ends had broken off some distance from the tip, and the new tip thus formed reached the sand. In some instances two such breaks had occurred. One of the test piles had not even penetrated to the depth of cut-off for the foundation piles to be driven later. Some feet in length

above the tip showed a mass of fibers, resembling worn-out rope, of about the same shape and size as a barrel.

The effect of continuing to drive temporary piles after the tip reaches rock was clearly shown during excavation at a tunnel portal in New York. Piles were furnished to the foreman in charge of the excavation somewhat longer than necessary to reach rock at the elevation indicated by the borings, and he was instructed to drive them to rock. The foreman reported that he had driven the piles as far as possible without bringing the butts below the upper wales which they were intended to support, and that they had not reached rock, insisting that they had moved quite uniformly until he stopped driving. For a number of feet the lower ends of the piles were badly shattered and broomed as revealed by the subsequent excavation, although the heads were not broomed materially.

A contracting engineer of large experience has expressed his conclusion that more piles are dangerously injured by improper driving than are rendered unsafe through insufficient driving. Another engineer concludes that, in general, piles are apt to be overdriven and much of their value lost. The former believes that this is due largely to the use of drop-hammers with excessive weights and undue heights of fall. He reported a case where 10 successive piles in an important foundation were driven in hard material with a 3000-pound drop-hammer and a fall of 30 feet and after practically reaching "refusal" suddenly moved several inches at the head. The inspector in charge accounted for the resumption of penetration by assuming that the foot of the pile had met and passed through a layer of hard material. The subsequent removal of these piles showed that every one had been broken near midlength. He also had frequent occasion in the removal of timber-pile piers to examine the piles after they were withdrawn and found that when driven to hard bottom they were generally either broken at a considerable distance above the foot, or else the foot had split and spread out so as to resemble an inverted mushroom several feet in diameter. He claimed, moreover, that excessive driving results in a degeneration of the fiber

which reduces the strength of the piles acting as columns and hastens decay materially.

A contractor, having noticed the apparently injurious effect on piles due to very heavy blows, ordered experiments to be made to learn just what damage was done and the proper remedy. Spruce and yellow pine piles 40 to 50 feet long, 12 to 15 inches at the butt and 6 to 8 inches at the tip were driven into a mixture of clay, sand, gravel and small cobble stones, which offered a gradually increasing resistance to penetration. A drop-hammer weighing 3000 pounds was used with the hoisting rope attached. The object was to determine what height of fall could be safely adopted without impairing the integrity of the pile either by brooming its head or its foot, or by breaking it at some intermediate point. The first piles were driven according to the former custom of raising the hammer to the top of the leads until the fall reached about 25 feet after which it was not increased. The fall was gradually diminished as successive piles were driven. Each pile was pulled up by a 100-ton derrick and examined. Nearly every pile driven with a fall exceeding 10 feet was found to be more or less injured, either by badly brooming at the foot or by breaking at some distance higher. A fall of 10 feet could be depended upon not to injure the pile. To make sure that the fall should not exceed this distance the instructions to the foreman placed the limit at 8 feet for a 3000-pound hammer, the driving to be continued until the penetration should not exceed 1 inch in the last three to five blows as circumstances might warrant. Extensive subsequent experience in pile driving under many other conditions confirmed the results of the experiments that a greater fall than 10 feet for a 3000-pound hammer is uncertain in its results and more likely than not to injure the pile.

The effect of a heavy drop-hammer, weighing 3800 pounds, is seen from the results in driving 34 piles for a temporary trestle at an undercrossing of a railroad near Columbus, Ohio, 13 piles, or 38 percent, being more or less damaged. Several were even telescoped, buckled and bent almost beyond belief, so as to be practically without value to sustain loads.

The following example illustrates conditions which too frequently prevail and lead to overdriving. No provision was made for an adequate exploration of subsurface conditions. It was known, however, that the soft mud was interspersed with layers of gravel hardpan of varying thickness. It was therefore decided to drive the piles so hard that it might be safely concluded that they rested upon a stratum thick enough to carry the required load. Ordinary spruce and hemlock piles split and smashed. Finally, Nova Scotia spruce piles full of solid knots were obtained, which were said to be so tough as to resist splitting, and yet soft and elastic enough to absorb the blows of the hammer. It was claimed that a satisfactory pile foundation was made with them. Unfortunately, the opinion prevails too widely that several extra blows at the end of the driving generally secure extra resistance of the pile or extra bearing capacity.

The limit of safe driving depends chiefly upon the weight and fall of the hammer, the material penetrated, the species of wood in the pile, its diameter and length, and the protection given to the head of the pile while being driven. Whether the timber pile is dry or green or improperly creosoted will also need consideration. Two piles have been driven side by side through the same ground and under the same conditions, so far as this is practically possible; one pile penetrated to its full length without apparent damage while the other "burst all to pieces," the difference in behavior being due to the fact that the first pile was sound while the second one had incipient decay called "red heart." The pile which failed appeared to penetrate the ground rapidly and easily, but the head showed signs of distress at an early stage of the driving. The elements affecting safe driving are discussed further in Arts. 29, 30, 31, 35 and 37. Another fruitful reason for overdriving piles is the unreasonable specification that is frequently adopted which requires driving to refusal. This is considered in Art. 40.

There is danger from overdriving when the hammer begins to bounce. Overdriving is also indicated by the bending. kicking or staggering of the pile. When a pile has not penetrated

very far and the hammer begins to bounce and the pile to shiver and spring near the ground, it is time to stop driving, unless the pile is disproportionately long for its diameter. In fairly homogeneous ground, if the driving becomes hard and the hammer starts to bounce it is usually wise to stop driving. If driving is continued and the rate of penetration is irregular, it is probably safe to assume that the pile is either brooming at the tip or fracturing at some intermediate portion of its length. If a pile suddenly changes direction, there is but little doubt that it has broken. In general, when a pile is sinking easily and suddenly stops, and the hammer commences to bounce, the driving should cease, as it is probable that the pile has struck a boulder or some other obstruction. The quality of a pile can usually be judged by the behavior of its head under moderate driving. As the driving progresses, the condition of the head also gives some indication regarding the action of the pile below the surface.

The best measures to prevent injury to piles by overdriving include the use of a cap to protect and guide the pile head; the substitution of the steam-hammer for the drop-hammer; the use of the water-jet whenever practicable; and an adequate exploration of the ground to be penetrated. The steam-hammer is more effective than the drop-hammer in securing the penetration of a pile without injury, because of the shorter interval between blows. Some piles have taken over 1200 blows without any sign of the head being broomed. The use of the water-jet is one of the most effective means to avoid the danger of overdriving, since it reduces the resistance to penetration. Preliminary exploration of subsurface conditions is necessary to interpret properly the behavior of a pile while being driven, as well as to determine the proper length of pile and whether it is to act as a column, or to support its load by skin friction. In the preceding paragraph a reference was made to the significance of an irregular rate of penetration; it would be entirely different in stratified material where the pile continues to break through one thin stratum into another one of different density. If a hard crust has to be penetrated and

the piles cannot do so without injury, it is better to use dynamite to break it up, placing it by means of a pipe. Or, a double-strength wrought-iron pipe, with a steel point and cap, may be driven until it penetrates the hard stratum, and after withdrawing it the timber pile may be inserted in the hole and driven to the proper depth. This method has been used to penetrate a hard embankment under railroad tracks. In other cases the hard overlying stratum may be removed by dredging or otherwise, and replacing the material if it is needed to provide lateral resistance. Attention is called to the references on this topic in Chap. XIX, especially to the Proceedings of the American Railway Engineering Association. It is often remarked that judgment based on experience will dictate when to stop driving. The training of the judgment depends not so much on the amount of experience as upon the habit of careful reflection on the results of observations in pile driving and of the probable causes in each case. It is most earnestly to be hoped that the time will come soon when it cannot be truthfully said that "the most prevalent bad practice in pile driving is overdriving" (see Fig. 19*a*).

ART. 20. SPACING OF PILES

In good practice timber piles are never spaced closer than 2½ feet between centers, and preferably not closer than 3 feet. When piles are supported by frictional resistance they should be driven so far apart, or to such a depth, that the increased area of bearing developed by the conoid of pressure, which has the required altitude to contain the friction resistance, reaches a level whose material will afford the required support before it intersects the corresponding conoid of an adjacent pile. This indicates that the character of the ground at different depths should be known before the number of piles is determined and any of them are driven, or else the number required to support the given load must be changed and hence the spacing required.

When an important function of the piles is to compress the ground penetrated by them a closer spacing may afford larger

bearing power at a given level, while, on the other hand, in some kinds of material one or more extra piles in a small group may reduce the supporting power at the same level. If piles are spaced too close together, the entire mass of earth enclosed by the group tends to sink as a unit.

In discussing this subject Goodrich states that "the best practice is to assume a given load per pile, to design all footings accordingly and to require the superintendent of construction to provide and drive piles which will sustain this assumed load. In that case the designer's care will be to provide just the proper number under each footing and to space them so that each pile will develop its full proportion of the given load. To this end, groups should be made as nearly circular as possible, especially when they consist of any considerable number of piles. The corner piles of square groups of 16 piles might just as well be omitted. It is of the utmost importance not to space piles too closely together; or if close spacing is necessary, to drive them all to such a depth that the bearing power of the earth at that depth is sufficient to provide the necessary support. All the piles under a building should be driven to the same depth, if possible, and the areas of groups should be carefully proportioned to the loads to be carried, unless the spacing is large enough for each pile to develop its full supporting power independently. Tests made by the Department of Docks and Ferries of New York City prove conclusively that piles driven in North River mud, even to considerable depths, influence each other to some extent when 6 feet apart, and are practically a unit in their action when only 3 feet apart. A group of two piles thus spaced had a supporting power only about $1\frac{2}{3}$ times what a single pile developed when properly spaced.

"Earth with 35 percent of voids, if compressed so that all voids are filled, will increase in density only 54 percent. From quite a number of tests of the compressibility of soils made by the writer, it is evident that a tremendous amount of energy is wasted in pile driving if the piles are spaced so closely that any great compressing of the soil must be done. This wasted energy is not disclosed in any pile formula, and serves to give

exaggerated values when such formulas are applied. Considerable practical experience also confirms this and all the other theoretical results given above. Thus, it is evident that, even with piles spaced 2½ feet apart, the amount of compression suffered by the earth is more than one-quarter of the maximum possible amount in many cases and that considerable energy must be wasted in driving so closely. A spacing of 3 feet is much to be preferred, especially when it is seen that the theoretical depths to which it is necessary to drive the piles, in order to develop a safe bearing power of 40,000 pounds, are 16 feet for the 3-foot spacing and 26 feet for the 2½-foot spacing. The writer [GOODRICH] thinks that a minimum spacing of not less than 2.7 feet should ever be allowed and that 3 feet should be used whenever possible."[1]

In discussions on pile driving one may find such examples as that in which 100 timber piles 60 feet long and 12 inches in diameter at the butt were driven into soft material, 2 feet apart each way, to sustain a static load of 600 tons, followed by the remark that no settlement was observed. In this connection it may be well to quote the following statement by WELLINGTON: "Bearing piles should be spaced at least 3 feet center to center each way if this gives a sufficient number to carry the load, and they are worse than wasted if driven less than 2½ feet center to center."

Bearing piles may be located in plan either at the vertices of a series of squares, or of a series of equilateral triangles. When a considerable area is covered, the piles are located thus in parallel lines on the plan, being opposite to one another in one scheme, and staggered in the other. Where the piles are staggered, the alternate piles in alternate rows are sometimes omitted over a part of the area instead of slightly increasing the spacing throughout some rows. As stated in a preceding paragraph, groups of piles under column footings should be arranged in plan as nearly circular as possible (see Fig 163*d*).

[1] Transactions of the American Society of Civil Engineers, vol. 54, page 448, June, 1905.

Since it is not always an easy matter to hold a pile closely to line, it is of especial importance to use range boards and transits when necessary to line up the leads of a floating pile-driver, so that the pile may be started in correct position. Telescope leads may be used to hold piles against the action of the current until the penetration is sufficient to hold them. Piles which are so crooked that they cannot he held fairly to line should not be used. The stresses which are produced in some piles in trestle bents or other structures by forcibly bending them, before the cap and bracing are attached, are often so great that their supporting power as columns is considerably less than that for which they were designed.

ART. 21. CUTTING OFF AND REMOVING PILES

When the heads of piles are to be embedded in a footing of concrete, it is unnecessary to have them cut off at exactly the same level; in fact, it is often specified that a certain proportion of them shall be cut off at a higher level than the rest. The heads should be cut off approximately level in this case but great precision is not required.

When timber caps or timber grillage, however, are to transfer the load from the substructure to the piles, it is important to cut off the piles at the elevations marked on the plans and that concave, convex or inclined heads will not be accepted. In the open air the cut can be made by an ordinary cross-cut saw upon two straight-edge guides attached to the piles. The next best method is to use a circular saw mounted upon a vertical shaft which is rigidly supported by a movable frame.

The piles should be cut at such an elevation that the top of the timber grillage is below the ground water at its lowest stage. Changes in this level due to probable changes in the drainage system should receive due consideration. Where tide water has access to piles, it is often customary to keep the timber of the foundation below half tide rather than below low tide, since it will be kept wet continuously by the rise and fall of the tide. The same conditions hold with respect to the

heads of piles embedded in concrete, otherwise they will suffer from dry rot.

When timber piles have to be cut off below the water surface to a given elevation, special care is necessary as well as properly designed equipment. At Portland, Ore., 256 piles for the pivot pier of the Morrison Street bridge were cut off as close as possible to the bed of the river. The rig designed for this purpose consisted of a carriage running on tracks supported by solid falsework. On top of this was placed a second carriage working across the first one. Suspended from the upper carriage was a four-post steel frame built of angles and rods, which extended down to the required depth, and upon one corner carried the shaft to which a 5-foot circular saw was attached. The saw was operated by an electric motor. By careful operation the piles were cut off to a practically true level.

At the Cambridge bridge over the Charles River the contractors used a heavily constructed machine to cut off foundation piles from 15 to 34 feet below the water surface. The scow supported regular pile-driver leads 60 feet high. The saw was 42 inches in diameter and attached to a 4-inch hollow shaft, the bearings of which were supported by a spud or vertical timber 14 inches square which could be easily raised or lowered between the leads. The driving pulley occupied a fixed position and engaged a continuous spline or key attached to the shaft. The saw was operated at a speed of 400 to 500 revolutions per minute by means of a 40 horse-power engine and a boiler of still larger capacity. The usual rate of cutting 10-inch spruce piles was 600 to 800 per day of 10 hours, with a maximum of 600 in a half day. Horizontal range sights were established and lines painted on the spud to determine the proper elevation of the saw as the tide changed. This method has been used in cutting off piles 60 feet below water surface.

The Department of Docks and Ferries of New York City has an equipment especially designed and constructed for rapid and economical operation. It has cut 115 piles 14 inches in diameter in 5 hours, and a maximum of 11 piles in 7 minutes. Its capacity is only limited by the ability of the crew to remove

the butts. The special engine has a vertical shaft, double-acting cylinders, cranks set 180 degrees apart and a 5-foot fly-wheel. The engine operates at 300 and the saw at 1000 revolutions per minute. The driving pulley has a key, engaging a seat cut 25 feet long in the 3½-inch saw shaft, which is 34 feet long and has its bearings bolted to a spud 52 feet long suspended between the leads. The saw avoids danger of binding or jamming by cutting off a pile before it can be stopped by an ordinary obstacle.

At Superior Entry, Wis., where foundation piles were cut off 2 feet above the bottom of the lake and 35 feet below low water, it was necessary to secure accurate cutting to grade to provide a uniform bearing for the timber cribs. The piles had been driven butt down without extension leads and a follower, and hence their heads projected above the water surface. In order to hold the saw at the exact elevation, a guide bracket was attached to the saw shaft whose trolley with double-flanged wheels took bearing upon an 8- by 8-inch timber cap temporarily placed on the adjacent line of piles. To cut the last row the cap was placed on three piles driven for this purpose. Errors discovered by measuring the lengths of cut-off were limited to ¼ inch. With good weather and quiet water the machine frequently cut 45 piles per hour, and occasionally a bent of 10 piles was cut in six minutes, the diameters at the cut being 11 to 19 inches. The crew consisted of seven men on the scow and five men who removed the parts cut off and placed the temporary caps in position. Allowing two hours to transfer and set up the machine, in addition to the actual time for cutting 230 piles, the work was done at an average cost of 13.75 cents per pile.[1]

Where only a small number of piles have to be cut off or in locations where equipment like that previously described cannot be used on account of interference with structures, a simple device may be adopted and operated by hand. A rigging used on the Chicago and Northwestern Railway consists

[1] For additional details and illustrations see Engineering News, vol. 63, page 696, June 16, 1910.

of a triangular frame in which a saw 4 feet long is held between the ends of a bent iron bar 2 by ½ inches in section which forms the other two sides of the triangle, each 8 feet long. The frame is suspended at its apex from a stick fastened to the lower surface of the stringers of a pile trestle, and operated by hand near the water surface. Another device consists of a vertical frame formed by two timbers crossing each other like the letter X; at the crossing a pin is driven into the pile to be cut off; the saw is held between the lower ends of these timbers and the upper ends are braced by a horizontal timber bolted on just below the handles by means of which it is operated.

At New Orleans a floating pneumatic caisson has been employed to cut off the piles and to construct the timber grillage upon them for terminal piers in the harbor. The caisson is surmounted by a tank for water ballast, and is partly supported by two barges on its sides which are rigidly connected by a framework of Howe trusses. The lower edges of the working chamber were submerged from 15 to 18 feet. The construction of the pneumatic caisson is described and illustrated in the Engineering Record, vol. 54, page 125, Aug. 4, 1906.

Piles are often used for temporary construction, such as to support falsework for the erection of a bridge, and have to be removed afterward. If its penetration is not too large, a pile may be pulled by the pile line of a pile-driver or with the aid of block and tackle. To reduce the initial resistance, the pile should be tapped by the pile-hammer before pulling it; or if the water-jet equipment is available it may be used to loosen the pile so that it can be easily removed. In tide water, piles are sometimes fastened by a chain to a scow at low tide and thus pulled by a rising tide. If hard ground surrounds a pile it may be started by securely spiking a block of wood on each side and lifting it by the aid of two screw jacks.

To remove the falsework piles of the Municipal bridge at St. Louis, a heavy timber tower formed of two bents battered in both directions and thoroughly braced was constructed on two barges. "The barges were placed about 10 feet apart in

the clear so as to straddle the double line of piles forming a bent. Two sets of falls, composed of four-sheave steel blocks, reeved with wire rope, were used and attached to the pile by means of chains. After the pile was lifted about 20 feet by the main falls, they were disconnected and the pile lifted clear by means of a runner passing through a snatch block attached to the lower chord of the bridge span. From 30 to 45 piles per day of nine hours were pulled with this rig."

In excavating for a foundation on reclaimed land in Kansas City it was necessary to pull some old piles which averaged 18 inches at the butt, and 40 feet in length. The rig employed consisted of four triple and four double blocks in combination, the 1⅝-inch hoisting line being run to a 25-horse-power hoisting engine. A water-jet from a 7–5–10 duplex pump was also used. An illustrated description of a sweep pile puller and of a tripod pile puller to be used on land or in very shallow water together with a statement of costs may be found in Engineering News, vol. 49, page 338, Apr. 16, 1903.

Another method of removing piles as an obstruction in a waterway is to cut them off with dynamite. A hole about 1½ inches in diameter is bored down along the axis of the pile with a ship auger and one or more sticks of dynamite inserted and exploded. In some cases 75 percent dynamite has made a clean cut. In one instance where a foreman was instructed to use 70 percent dynamite he used 40 percent dynamite, as that was more easily obtainable. As a result the piles were merely shattered and not cut off. Later, on using two ½-pound sticks of 70 percent and one of 40 percent dynamite the largest timber pile was cut off and the top hurled over 50 feet into the air. The holes bored were 4½ feet deep, and the cost was 55 cents per pile for labor, dynamite, fuse and cap. Dry sand may be used to fill the hole after inserting the dynamite, but does not need to be tamped.

In a report on the removal of a temporary pile bridge to clear the river for floating ice and logs in the spring, 40 percent dynamite was stated to be effective. A ring was formed of telegraph wire which was large enough to slip over a pile, three

half sticks of dynamite were fastened equidistant on the wire with a percussion cap in each. A fuse was attached long enough to reach the bottom of the river when the wire was dropped over the pile, and was connected to a battery. All the piles were cut off clean at the bottom, making this method the cheapest and quickest way to remove the obstructing piles.

To remove a cluster of 13 large pine piles at Leavenworth, Kan., which had been sunk by a water-jet, and could not be pulled on account of high water and floating ice, a 3-gallon jug was placed in hot water and partly filled with hot sand so as to store as much heat as possible. The remaining space was filled with 30 pounds of dynamite. After arranging an exploder and battery, the jug was corked and lowered through a small opening in the center of the cluster and on reaching the sand bottom at a depth of 14 feet it was exploded, thus cutting off the piles at the level of the jug.

ART. 22. MARINE BORERS

Marine borers belong to two families, the Crustacea and the Mollusca. The three genera of the Crustacea which are destructive to timber are the Limnoria, Chelura and Sphaeroma. Limnoria resemble the ordinary wood louse, having bodies from 1/8 to 1/4 inch long. They attack timber by running shallow burrows just beneath the surface of the wood. Because of the large numbers frequently 200 or more to the square inch of timber, they quickly destroy the outer layer of wood, which washes away, exposing the next layer to attack.

Limnoria attack piles mostly between an elevation slightly below low tide and about half tide, although they may work down to mud line and sometimes above high tide. This borer is widely distributed on the American coasts, extending from the Gulf of St. Lawrence to the Falkland Islands and from Alaska southward an equal distance. The limits of necessary salinity of water have not been definitely determined, but this borer has not been found where the salinity falls below 15 parts per thousand for a considerable length of time. Sphaeroma have been found in practically fresh water.

Mollusca are bivalves distantly related to the clam family, the most important genera being the Teredo, Bankia and Martesia. The latter resembles the clam in general appearance, being wholly enclosed within a shell, but the Teredo and Bankia have worm-like forms with heads of shell material.

Mollusca enter timber when very small and spend there their entire life, burrowing and enlarging their holes as they grow. A pile may have only a few small holes exposed and yet be completely honeycombed on the inside. The Teredo is generally only a few inches long, while some species of Bankia reach a length of 3 or 4 feet and a diameter of an inch. Borings made by Martesia are generally not over 1 inch in diameter and 2½ inches long.

Where the waters are infested by marine borers, timber piles which are not chemically treated or mechanically protected have a very short period of usefulness. The average life of a timber pile on the coasts of the South Atlantic, Gulf and Pacific states ranges from about eight months to two years. The development and activity of the borers are stimulated by high temperatures, and hence in some of the more northern coasts the average life may extend to three or more years. The minimum life of service is considerably shorter. For example, on the coast of California, where the average life of a pile is estimated at about two years, a pine pile 15 inches in diameter has been eaten off entirely within eight months; and on the Gulf Coast in Texas, where the average life ranges from one to one and one-half years, piles have had to be replaced in within 8 to 30 percent of that time. In very salty water and during a hot season a pile 18 inches in diameter may be entirely honeycombed in three months. In the summer of 1924 some piles in San Franciso Bay were destroyed in two and one-half months. It is claimed that in the vicinity of Puget Sound, "a stick of timber, rough sawed, will last about eight months, a peeled pile will last a year, a pile with the bark on will last a year and a half, while a creosoted pile will last from 15 months to 15 years."

Almost no timbers of the temperate zone are immune from borer attack, although some are less subject to attack than

others, while some tropical woods seem to be quite resistant. Some of the species more or less immune are cottonwood, palmettos, mangroves and palms, Eucalyptus, Philippine woods, Australian turpentine wood, Greenheart, Angelique and Manbarklak.

Cottonwood has given good service under some conditions, but not so good under others. Palmettos, mangroves and palms are not totally immune, but give better records than northern timbers. The foregoing timbers, on account of their lack of strength, are not considered as structural timbers and so their use must be limited to light structures. A number of Philippine woods are resistant to marine borers. Australian turpentine and Greenheart have given very good service and yet have some records of failure. Angelique and Manbarklak contain particles of silica which are said to prevent entrance of borers.

ART. 23. CHEMICAL PRESERVATION

Numerous chemicals have been employed to impregnate timber. Such chemical compounds as copper sulphate and lead acetate are extremely poisonous to animal life, but they tend to become leached out by the action of sea water and so lose their toxicity.

Creosote impregnation has proved one of the most satisfactory ways of protecting piles. The durability of creosoted timber piles depends upon several elements. The timber must be of good quality, free from decay, and should have sufficient sapwood to take the requisite amount of creosote oil. Timber may be of such high grade according to standard specifications for structural timber as to be unfit for chemical treatment. The creosote oil must be high grade, with the proper chemical constituents and physical properties, and the artificial seasoning and chemical treatment have to be conducted so as to secure the required impregnation.

The artificial seasoning and treatment of material is a continuous operation and occupies from 24 to 36 hours. After the green material has been placed in the cylinder, the doors

are closed and steam is admitted into the cylinder until the required steam pressure is indicated by the gage. This pressure is maintained until the wood is thoroughly sterilized and the sap liquefied; the steam pressure is then relieved and both air and the remaining steam are exhausted from the cylinder by means of a powerful vacuum pump. The temperature in the cylinder during this portion of the treatment is kept above the vaporization point corresponding to the degree of vacuum, so that the liquefied sap is vaporized and passes off from the timber to the vacuum pump. After all the moisture has been exhausted from the cylinder and the wood is perfectly dry, the cylinder is filled with oil from an elevated tank. The oil pressure pump is then started and kept in operation until the oil in the cylinder is under a specified pressure. This pressure is maintained until the established system of measurement indicates that the timber has been impregnated by the required amount of oil. After relieving the pressure the cylinder is opened and the timber withdrawn.

When the treatment is carefully done and the full amount of impregnation with the best quality of creosote, or dead oil of coal tar, is secured, the timber will not be materially reduced in strength and it will be protected effectively from the Teredo and the Limnoria. At an inspection made in 1905 of over 4000 creosoted piles in the railroad pile trestle connecting Galveston Island with the mainland, which had been in service for 10 years, no traces of decay or deterioration could be found. An inspection made in 1913 after a service of 18 years showed over 90 percent of the piling good.

In 1904 and 1905 it became necessary to replace the truss spans of the bridges over East and West Pascagoula rivers, and of the Rigolets and Chef Menteur crossings on the New Orleans and Mobile Division of the Louisville and Nashville Railroad, by new girder bridges and truss swing spans to accommodate the heavier rolling stock. Examinations by boring showed so many of the creosoted piles which had been driven in 1876–78 to be still perfectly sound that it was decided to use them in the new structures. Intermediate pile piers were

driven and the old ones reinforced by additional piles, many of which were sunk by jetting. In the Chef Menteur bridge not a single pile was replaced on account of defects, and in the other three bridges the proportion was less than 10 percent. Of those rejected and replaced by new piles not one had been damaged by the Teredo, nor had the outer ring penetrated by the creosote shown any signs of deterioration, nor was a single pile seriously decayed. The rejected piles showed an interior dry rot which indicated that they were not perfectly sound when treated. As the new cut-off level was several feet lower than the old, and below the extreme high-tide level, there was opportunity for a thorough examination of each pile's condition.[1] These results are more favorable than can usually be expected. Extended experience shows, however, that creosoting may be depended upon to protect timber piles in gulf waters from 10 to 15 years, after which it is generally necessary to furnish additional protection by enclosing the piles with terra cotta, concrete or steel pipes, as described in the next article.

When this railroad was built in 1869–70, untreated piles and timber were used to build the trestles which cross numerous arms of the Mississippi Sound. Before trains had been running six months an engine was precipitated into Biloxi Bay where the green piles had been eaten by the Teredo. The unsatisfactory results obtained by mechanical protection of piles in other trestles whose piles were also attacked led to the decision in 1875 to rebuild with the creosoted piles referred to in the preceding paragraph. The piles were treated in a plant owned and controlled by the railroad company. Of 5093 piles driven in 1877–78, in 19 pile trestle bridges, 617 were still in service in 1908 as they were originally driven; on account of settlement, 868 were either redriven or were cut off to place framed bents upon them in the years 1904, 1906 or 1908; 95 were replaced from time to time, including 70 in three trestles which had been cut off by the Teredo; while 3513 piles were protected mechanically in 1892–93 in five trestles where the

[1] Engineering News, vol 61, page 277, Mar. 11, 1909.

action of the Teredo was especially severe, and in which the 95 piles had been replaced.

In some piling work in Portland, Maine, in 1922, where southern pine piles from 85 to 89 feet long were used, the bark was left on for the lower 30 feet, that part which would be in the ground after the piles were in place. It was thought that the presence of the bark would largely prevent the entrance of creosote oil where it was not needed.

Experience shows that, where marine borers have attacked creosoted piling, this attack always starts where the penetration of the oil is small or where the surface of the pile has been damaged, hence the necessity of full and uniform creosoting treatment as well as careful handling of piles.

The limitations of space prevent the insertion of specifications for the chemical preservation of piles, and a discussion of various methods of treatment, and of the tests to be applied to the oil, etc., and hence the student is referred to the Reports of the Committee on Wood Preservation of the American Railway Engineering Association, to the Proceedings of the American Wood Preservers' Association, to some investigations by the United States Forest Service and to Marine Piling Investigations, published (1924) by National Research Council. In these publications the discussions of the subject are kept up to date, and the results of good practice are recorded.

Art. 24. Mechanical Protection

Except for the Sphaeroma, marine borers are confined to salt water. The portion of the pile commonly attacked lies between mean tide level and about 4 feet below low tide. The extent of the ravages inside is not indicated by the position of the entrance holes. Experience has shown that any protection applied to the surface of the pile must extend from an elevation a short distance above high-water line to an elevation below the mud line, due consideration being given to the probability of a change in the mud-line elevation.

Since chemical treatment must be applied to the entire length of the pile, and is, accordingly, quite expensive, various methods

of mechanical protection have been devised, which are applied only to the vulnerable surface of the pile. This method has also been applied in conjunction with the use of creosoted piles. Figure 24 is reproduced from Circular 128 of the United States Forest Service, entitled Preservation of Piling against Marine Wood Borers, by C. STOWELL SMITH, to which reference may be made for further information.

Figure 24*a* illustrates a pile with the bark left on, which affords protection, provided the bark remains absolutely intact. This protection is due to the presence of certain oils and acids in the bark and also probably because of the fibrous structure of the bark, making attack more difficult. Extreme care must be taken in handling and driving not to break the bark. Where knots and breaks occur, a covering of sheet metal should be used.

In Fig. 24*b* the pile is sheathed with planks, a method formerly thought to be effective, but now recognized to be one of but slight value. If tar paper, felt or other similar material is used under the sheathing, or if the sheathing is creosoted, this method may be fairly effective.

In ancient times attempts were made to protect galleys by driving the hulls full of nails, and this method has been applied to piles as shown in Fig. 24*c*. It is not customary to completely cover the pile surface, as protection comes not only from the nail heads but also from the rust formed by the corrosion of the metal. The Sante Fé Railway has used this method with 3d. and 4d. nails spaced ½-inch centers. A method somewhat similar to the use of nails is the application of strips of sheet iron with open spaces between the strips. This is probably not so effective as the use of nails, as the rust does not penetrate the wood so deeply.

Figure 24*d* illustrates a type of protection where the pile is covered with burlap soaked in various chemicals. In some cases the surface of the pile is painted prior to the placing of the burlap. A number of these methods are patented. In one process the impregnating solution consists of asphalt, slaked lime, rock salt, sulphur, marble dust and sand.

Figures 24*e* and *f* illustrate the use of a metallic sheathing consisting of thin sheets of iron, zinc, yellow metal or copper. Iron is seldom used as it corrodes rapidly. Zinc sheathing, 9- to 14-inch gauge, prolongs the life of the pile but records do

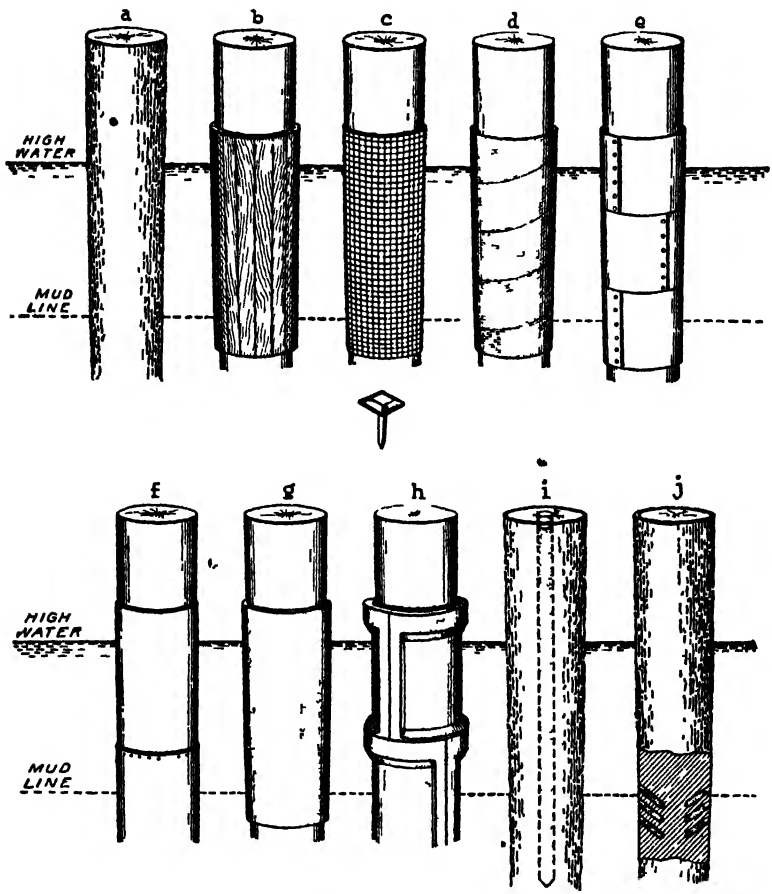


FIG. 24, *a* to *j*.

not indicate that it is an economical material. Yellow metal, an alloy of copper and zinc, has been widely used with considerable success. Copper sheathing is expensive and its high junk value makes it likely to be stolen. It is an effective protective

covering, but, like all other metallic sheathing, it is subject to abrasion and is easily torn by floating débris.

Figure 22g illustrates a casing of cement mortar or concrete in contact with the timber pile. The results obtained and the cost depend largely upon the design of the forms and the method of handling the concrete. In one case the forms are made of sheet steel in sections 18 inches long, and split longitudinally into halves. Vertical angles are riveted to these halves so that they may be clamped together with rubber gaskets between them to make tight joints. The lower end of each section is slightly reduced in diameter and fits tightly into the slightly enlarged upper end of the next lower section. The mortar consists of one part cement to two parts of sand and, by means of a double-end scuttle, is placed by a diver into each section of the form before the next section is placed in position. The lowest section is placed in an excavation in the bottom, the mortar is extended up to mean-tide level, and the remainder of the pile painted with a wash of neat cement. This method has produced good results, although it is more expensive than some others.

In another device which is patented, the vertical edges of the semi-cylindrical sections of galvanized iron are covered with compressible material and are held together on each side by a sliding clamp with a ring at the top. In operation, the sections are put together on top of each other near the head of the pile and gradually lowered until the form extends to the bottom and is pumped out. After the form is filled and the concrete set, the chain of clamps is pulled up by the rings, thus releasing the halves of each section, which are then hoisted by the attached ropes. Shells of concrete without reinforcement should not be placed around the piles until they have been long enough in place so that no further swelling will occur, otherwise it will crack the concrete.

A different method, which can only be used on new construction, consists in pre-molding a tapering reinforced-concrete shell long enough to reach from above high water to a short distance below the bottom, and lowering it over the pile by a hoisting line

of the pile-driver. The weight of the hammer, or light taps with it on a suitable cap, serve to sink the shell to its desired elevation. After wedging the shell in a concentric position about the pile head, the bottom is sealed with rich concrete and after two or more days the enclosed water is pumped out and the space filled with lean concrete. This construction has been used where the wharf superstructure is of reinforced concrete and the column reinforcement is carried down into the upper end of the annular space betwen the pile and the shell and into which a richer mixture of concrete is placed.

In the harbor at Seattle, Wash., a lot of piles were protected by a coating of cement mortar. After fastening a wrapper of poultry wire netting about each pile, the covering was deposited on the surface to a thickness of 1½ to 2 inches by means of a cement gun. The gun was operated between tides and the "gunite" set up so quickly that no trouble was experienced from the rising tide. A coating of gunite reinforced by longitudinal rods and wire fabric has also been applied to new piles over a part of their length, before being driven, to serve as a mechanical protection against marine borers (see Art. 49). In one case the coated lengths were less than 40 percent of the lengths of the piles. In another case ½-inch longitudinal wood furring strips were fastened to the pile and galvanized wire cloth wrapped around the outside. A 1:2 cement mortar sheathing was then troweled on the mesh to a minimum thickness of ½ inch over the mesh.

Another type of protection consists in slipping sections of vitrified clay pipe or of reinforced-concrete pipe over the heads of piles and filling up the intervening space with sand. To protect piles in this manner after the cap or superstructures are in place requires the use of pipe divided longitudinally into halves. Figure 24*k* gives the dimensioned plans of a patented concrete pipe of this kind known as the lock-joint pipe. The two halves are placed around the pile and locked together by inserting wooden keys, soaked in hot tar, in the scarf joints. The pipes are molded in iron forms and after seasoning are placed in position from a raft moored alongside of the piles.

Vitrified pipe is sometimes used in a similar manner, but as these halves have butt joints they must be wired together and it is difficult to get tight joints to hold the sand.

The use of natural bark, nails, and burlap soaked in various compounds for protection is unsatisfactory, since they are easily injured by floating débris or by impact from the waves. The artificial bark is attacked by the borers and must be replaced before it becomes so weak as to be knocked off by driftwood. The metallic sheathing is more expensive than chemical preservation for the entire length of piles and requires costly repairs. The sheathing of reinforced concrete has suffi-

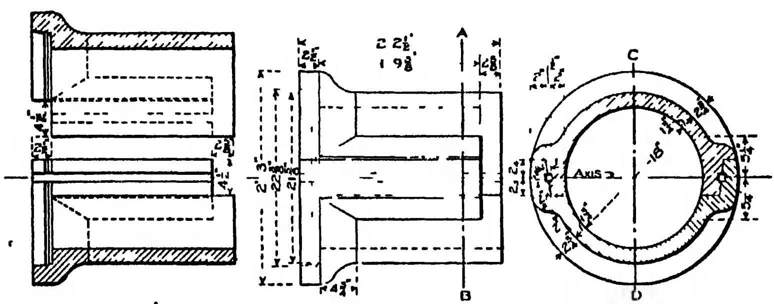


FIG. 24*k*.

cient strength to resist the impact of driftwood, and the mesh reinforcement usually holds it in place even if it should become cracked. Better results are obtained with concrete than with cement mortar. The mortar or concrete filled between the removable form and the pile mixes with the mud at the bottom to such an extent that the lower part of the protection frequently breaks off. It is impossible to prevent this entirely without considerable expense. A more serious fault is the cracking and breaking off of the upper part of the encasement. Scour at the bottom exposes the unprotected wood.

These defects are avoided by the use of pipe with a sand filling, as the pipe slips down and one or more sections can be added above when this is observed. If an intermediate section is broken, attention is called to it by the sand running out.

Repairs are easily made by removing the broken pieces, lowering the upper sections, adding a new section on top and filling in the sand as before. This method of protection has been successfully used for pile trestles on some railroads since 1893. Reinforced-concrete pipe is superior to vitrified clay pipe on account of its greater resistance to accidental or other blows. In unusually exposed situations cast-iron pipes have been used. When creosoted piles have been standing in salt water so long as not to be immune any more from marine borers, they may be protected mechanically to prolong their service.

Figures 24*i* and *j* illustrate the protection of piles by boring holes into them and filling the holes with a poisonous substance. This treatment has been discontinued. For additional details the student is directed to the references on this subject in Chap. XIX. See also the combination piles which are described in Art. 49.

ART. 25. COST OF PILE DRIVING

So many elements enter into the cost of driving piles that it is difficult to give costs that are of real value in estimating unless the record is more complete than is customary in practice. Some statements of the cost of driving piles for the foundations of buildings, trestles, docks, bridge piers and abutments are given in handbooks of cost data and in engineering periodicals, but they can only be used with extreme caution since the local conditions and the methods of doing the work are usually not given in sufficient detail, if they are given at all.

The time and cost depend upon whether a steam-hammer or a drop-hammer is employed; whether a suitable cap is used to protect the pile head and guide its movement, or merely a pile ring; whether the water-jet is employed to sink the piles, or to aid the hammer in driving; whether the foot must be protected by a shoe; whether the piles are driven below the water surface by means of a follower; whether extension leads are used; whether the pile-driver must be moved over the timber bracing of an excavation, or directly on the surface of the ground;

whether the piles are creosoted or not, long or short, of hard or soft timber; driven with the butt or tip down, vertically or on a batter. The cost also depends upon the type of pile-driver, the magnitude of the job and the organization and experience of the crew; but especially upon the subsurface conditions, whether the driving is easy or hard, and whether special precautions are needed to avoid overdriving.

The following is a brief summary of driving piles for 41 trestle bridges averaging 101 feet in length on the Omaha and St. Louis Railway in the late fall of 1889. In 46 days 1267 piles were driven, ranging in length from 14 to 52 feet, the average being 24 feet. The penetration varied from 10 to 18 feet, or an average of 14 feet. The working time each day with the leads of the track driver in position averaged 6 hours and 32 minutes, and the time to raise and lower the leads, 14 minutes. The average time required to drive a pile was 15 minutes and to raise or to lower the leads, 2.5 minutes. The average cost per linear foot of the piles was 15 cents, and the average cost per pile in place was $5.14. The cost of labor for the 46 days was $1683.72; and for fuel and supplies $262.28, or 15.6 percent of that for the labor only.

The following gives an analysis of the cost per pile for the 4383 piles in place which support the timber grillage and masonry of Fort Montgomery on Lake Champlain, the piles being driven in 1844–46: net cost for machinery, $1.22; cost of piles, $1.40; driving, 40 cents; measuring, hauling, securing for winter and sharpening, 18 cents; pile rings, 10 cents; cutting off piles to receive grillage, 11 cents; net cost for other machinery than pile-drivers, 4 cents; contingent services and contingencies for this part of the construction, 43 cents; total, $3.88. Such an analysis in still valuable although prices have changed.

In the spring of 1902, on the Chicago and Eastern Indiana Railway, 436 piles were driven varying in length from 14 to 42 feet, aggregating 10,535 linear feet at a total cost for driving of $466.35, or of 4.4 cents per linear foot. In 1906, on the Chicago, Milwaukee, and St. Paul Railway, the average cost of driving foundation piles on 150 jobs was $2.45 per pile. In

different classes of work the cost ranged from \$0.75 to \$7.15; for piers and abutments, \$3.84. This variation shows the effect of differences in local conditions, including the size of the job.

In building the railroad trestle to the Sandy Hook proving ground, the cost of which was about \$10.60 per linear foot of track for a length of 4494 feet, the cost of driving creosoted piles was about 15 cents per linear foot. Three land and two water drivers were employed with drop-hammers weighing 1800 to 3000 pounds. The averages for several hundred piles observed were: fall, 14 feet; number of blows, 175; time, 20 minutes; penetration per blow, 1 inch. The minimum total penetration was 15 feet, and when the penetration was over 20 feet the water-jet process was used. The piles were driven during the last quarter of 1904.

In the construction of the viaduct approaches of the Southern Railway over Chattahoochee River, completed in 1907, the cost for the piles under the viaduct footings was found to be as follows: 13,750 linear feet of creosoted piles, \$3812.50; pile shoes, spikes and rings, \$352.33; coal, oil, waste, rent of driver, etc., \$758.72; labor for pile driving, \$3186.31; labor for sharpening piles, \$83.70; making the total cost per linear foot 59.6 cents, which includes 27.7 cents for the cost of the creosoted piles. The cost of freight and of train service is included in the items for materials, etc. "The high cost of pile driving was due to the fact that one row of pedestals came beneath the old trestle and thus required considerable manipulation of the driver and loss of time in working around the bents. The actual labor of driving piles for the outside row of pedestals was only a little over 9 cents per linear foot of piles in leads." In some estimates of cost, the cost of driving is taken from 65 to 100 percent of the cost of untreated piles.

CHAPTER III

BEARING POWER OF PILES

ART. 26. PILES ACTING AS COLUMNS

When piles on land project some distance above the surface they are usually held in position laterally by diagonal bracing whenever there is sufficient room for it so that the pile is not subject to direct bending. An example of this construction occurs in pile trestle bents. If the vertical distance between points of connection for the bracing is large, the pile must be designed to provide for column action.

Piles driven in water are frequently not braced and hence it is essential to design them with regard to their strength as long columns, since this may be the limiting condition rather than the bearing power of the ground penetrated. If the substructure placed upon the piles is not held laterally except by vertical piles, then the piles act like columns with the upper end practically free and the lower end fixed at some elevation which depends upon the material penetrated. To determine this elevation is the principal problem. Usually, it cannot be taken at the bed of the river or lake, since the material there is soft or yielding. Even if the bottom consists of firm gravel and sand, the required elevation is 1 or more feet below its surface. When the material is mud or silt, grading slowly into more compact material, the upper strata give relatively little resistance to the lateral deflection of the pile. That the resistance of such material is greater than may be naturally inferred from its consistency is proved by the fact that at New York piles frequently break off in case of trouble at approximately the mud line of North River silt.

It has been proposed[1] as a reasonable assumption to consider the lower third of the softer strata, which overlie the hard

[1] See Engineering News, vol. 60, page 18, July 2, 1908.

stratum, to be effective in lateral resistance and to ignore that of the upper two-thirds. This makes the assumed free length of the pile column equal to the distance from the pile cap to the river bottom plus two-thirds of the penetration in distinctly soft ground. The strength of such a column is equivalent to that of a column of double the length with both ends round. It must also be remembered that the strength of a group of pile columns has only the strength of one column multiplied by their number, since there is no provision made to resist their movement longitudinally with respect to one another. In this respect they are analogous to composite posts. (See JACOBY's "Structural Details," Arts. 49 and 50.)

It frequently happens that bearing piles transmit nearly or all of their vertical load to a hard substratum, overlaid by softer material, which will yield laterally under pressure. In all such cases the piles should be designed as columns.

If the foot of a pile bears on rock and the overlying material is not able to resist its lateral displacement it may be necessary to drill shallow holes into the rock. In shelving rock this method of preventing displacement is of especial importance. Sometimes riprap is used for this purpose, but its uneven weight upon the material overlying the rock has been known to cause sliding with disastrous results.

The section area of the post should be large enough to provide adequate bearing area and the taper of the timber pile should be as small as possible. In some cases it may be desirable to place the butt at the foot of the pile. The foot should not be pointed unless it is necessary to secure penetration for a short distance into material like hardpan, to prevent its lateral displacement.

The greatest care should be taken in driving piles which are expected to rest in bed rock or penetrate slightly into hardpan in order to prevent shattering or crushing the feet of piles, otherwise their supporting power may be seriously impaired. When the overlying material is muck or silt, or soft, yielding material, it is preferable to omit the resistance, if any, due to skin friction in designing the pile.

The conditions described in the preceding paragraphs often apply to falsework piles used to erect bridges. In any case it is desirable to brace the piles effectively by means of sway bracing, but in rivers which carry a large amount of driftwood during flood seasons, or large masses of floating ice, it is essential to provide the piles with carefully designed sway bracing and to add lateral bracing. Experience amply justifies special caution in design and construction for this purpose.

The unit-stress in the outer fiber which may safely be allowed depends upon the species of wood as for ordinary wooden columns, but some reduction is usually made on account of the piles being water-soaked. Sometimes its value is reduced by the use of a lower grade of timber, which has more knots and other defects than are permitted by specifications for structural timber. When no account is taken of the species of wood, specifications sometimes give the working unit-stress in the outer fiber as 600 pounds per square inch, reduced for pile columns to $600(1-l/60d)$, in which l is the unsupported length in inches and d the diameter at the middle of the unsupported length (see Art. 40).

When piles act as columns or are subject to bending moment especial care should be taken to drive them accurately in position; for, if they have to be forced laterally into line, account must be taken of the initial flexural stress thus produced.

In case it is necessary to deposit riprap to give lateral support to piles or to prevent scour it is better practice to deposit the riprap first and drive the piles through it, because filling in afterward has been known to bring such great lateral pressure upon piles as to cause their failure by bending.

ART. 27. THE GOODRICH FORMULA

The most elaborate attempt which has been made to deduce a general theoretical formula for the final resistance of a timber pile when subjected to the blow of a drop-hammer is that of ERNEST P. GOODRICH, the results of which are contained in a paper entitled, The Supporting Power of Piles, published in the

Transactions of the American Society of Civil Engineers, vol. 48, page 180, August, 1902. The phenomena of pile driving which are taken into account mathematically in deducing the formula, in accordance with the principles of physics and mechanics, are those described in the fifth paragraph of Art. 5.

It is then shown how 14 other pile-driving formulas may be derived from this general one, by stating the various assumptions with respect to its elements or terms which are made in each case. That some of the assumptions are seriously in error is proved conclusively by the wide variations in results obtained by the application of the formulas. The true values of some of the terms can be determined only by experimental investigation.

The general formula consists of 25 terms besides several numerical coefficients and exponents, and is therefore too complicated and unwieldy for practical use. For this purpose, a number of terms were evaluated with the aid of experiments conducted under proper conditions for pile driving in good practice. One of these is referred to in the sixth paragraph of Art. 5, and another in the third paragraph of Art. 30. By substituting the values thus obtained, and inserting suitable numerical values for the dimensions and weights of the pile and hammer, an expression was derived giving a direct relation between the pressure on the head of the pile when it comes to rest, and the penetration. From this relation, it was found that for an allowance of 3 percent error in the observation (which, for example, is a variation of 1/8 inch for a penetration of 4 inches), the corresponding error involved in the pressure on the pile is 3.1 percent when the penetration is 4 inches and 23 percent when the penetration is 1 inch. Hence, any terms in the formula which involve a change of less than 3 percent in the pressure on the pile may be advantageously omitted, and no penetration much less than 1 inch can be trusted to give the corresponding pressure within a reasonable percentage of error.

An extreme variation in the elastic shortening of the hammer is found to produce a change of only 0.07 percent in the pressure

on the pile, and hence the four terms relating to the deformation are omitted. The difference between the elastic shortening of a long softwood pile and that of a short hardwood pile may cause an extreme variation in the pressure on the head of the pile of about 25 percent, and hence this term is retained.

After introducing the experimental values, and making the other changes mentioned, the formula for the final pressure on the head of the pile, as it comes to rest, is reduced to

$$F = -\frac{p}{C} + \frac{1}{C}\sqrt{p^2 + 1.15CW_h h(R_w - v')} \qquad (1)$$

in which p denotes the penetration of the pile under a single blow, C the elastic shortening of the pile due to longitudinal compression, W_h the weight of the hammer, h the fall of the hammer, R_w the ratio of the weight of the hammer to the combined weight of hammer, pile and earth moved in connection with the pole and v' the ratio of the work done in crushing and heating the head of the pile to the total work done by the hammer exclusive of losses before it strikes the pile.

The coefficient 1.15 in this expression relates to the velocity of the hammer, it being found by experiment that, when the hammer is operated in the customary manner with the line from the engine attached to it, $v^2 = 1.15gh$, instead of $v^2 = 2gh$ for a free fall (see Art. 30).

This loss of energy may be computed by equation (1) from two sets of observations on the same pile for falls of the hammer which do not differ widely, provided it be assumed that both the total pressure or resistance of the pile and the loss of energy are the same in the two cases. From observations made on a number of piles GOODRICH found that the loss of energy v' rarely exceeded 5 percent and in most cases was nearly 2 percent for piles that were sound and well driven. From a given numerical example in which $W_h = 3000$ pounds and $h = 180$ inches, the value of F is found by computation to be 134,400 pounds when $v' = 0$, and 124,800 pounds when $v' = 5$ percent, or a reduction of about 7 percent. Without such observations and computations, it is absolutely impossible to form any reasonable judgment of the value of v', or of its effect.

To eliminate the value of C in equation (1), the same data are used and the values of v' computed from the formula but with C omitted. The value v' thus obtained for each pile includes losses due to the compression of the pile, as well as to heating and crushing its head. The values thus found vary greatly but average less than 10 percent, even with some very badly broomed piles. By plotting the percentage of energy losses due to all causes for the different falls of the hammer used in the experiment, the curve shows that the loss of energy increases with the height of the fall. The author of the formula states, however, that his observations tend to show that the terms involving the compression of the pile can be neglected and proper compensation be made by taking v' as 2 percent in the formula, provided the piles are sound and well driven; but the formula is liable to be in error about 20 percent, if the piles are poorly driven and the fall is much less than 15 feet.

By making these further substitutions, the formula becomes $F = 0.575W_h h(R_w - 0.02)/p$. The term R_w involves the unknowable quantity W_g or the weight of the ground moved in connection with the pile. It was estimated that for piles 700 inches long and weighing 2000 pounds, W_g should not be taken less than 1000 pounds, this estimate being based on observations of minature piles driven in a box of sand with glass sides, and of the ground found clinging to actual piles withdrawn from the earth. In special cases, such an assumption may involve an error of 33 percent and, if combined with other cumulative errors, the final value of F given by the formula may be 50 percent in error. The opinion was expressed by its author, however, that if a sound well-driven pile weighing somewhat less than the hammer be tested by a fall of about 15 feet and shows a penetration of about 1 inch; the formula in its final shape will give the supporting power of the pile immediately after driving, with a probable error of considerably less than 10 percent. Inserting the value of $R_w = 0.5$, the formula finally reduces to the expression $F = 0.276\ W_h h/p$, or by changing the height of the fall from inches to feet, it becomes

$$F = \frac{10W_h H}{3p}, \tag{2}$$

in which F denotes the ultimate bearing power in pounds immediately after driving, W_h the weight of the drop-hammer in pounds, H the restrained height of fall in feet, the line being fastened to the hammer and p the final penetration per blow, expressed in inches.

GOODRICH recommends "that in making tests for the supporting power of piles, a standard fall of hammer be adopted and specified for making all determinations, and that 15 feet be adopted for the following reason: (*a*) This height of fall produces good observable penetration with any but very light hammers, or for piles in extremely compact soils; (*b*) the penetration is not excessive for any but very heavy hammers or for piles in very light soils, (*c*) all frames are large enough to afford this fall; (*d*) the lost energy is comparatively small; (*e*) nearly all formulas give nearly the same values through this region of variation; (*f*) the writer's formula is especially built for this fall." After recommending a specification relating to the weight of hammer, height of fall and final penetration, he adds "that designers can more easily determine the necessary pile spacing and the most desirable factor of safety to be used in individual cases, and make the pile-drivers follow a standard specification, than otherwise"

The description in this article is given at such length because it properly emphasizes certain phenomena of pile driving which are often not fully appreciated, and in order that every one who uses the Goodrich formula may know all the elements involved in its deductions, the values determined by experiment, the methods of determining certain approximations, the practical basis for certain assumptions, the relative effect of different elements upon the bearing power and the general limitations under which it may be used properly as recommended by its author. If, for example, someone should pay no heed to these limitations, and proceed to substitute a value of zero for the penetration p, an infinite value would be obtained for the ultimate bearing power F, which is manifestly absurd. (See the discussion on the final penetration per blow in Art. 31.)

It must not be forgotten in this connection that all formulas for bearing power are deduced under the fundamental hypoththesis that the material of which the pile is composed can transmit a load of this magnitude through its head and at least a part of its length. Therefore, the strength of the pile under longitudinal compression invariably limits the load which it can support, if this value is less than that given by the formula.

ART. 28. ENGINEERING NEWS FORMULA

The Engineering News formula for pile driving was developed by A. M. WELLINGTON in an approximate and simple manner as compared with that of GOODRICH, by considering the subject more from a purely practical standpoint. The work done by the hammer having a weight W, in falling freely through the height h, is Wh. The useful work done upon the pile is the product of its resistance multiplied by its penetration under the last blow. The ratio of these two products measures the efficiency of the hammer blow, and depends likewise upon the proportion of work wasted. The penetration was called the "set" by WELLINGTON and hence designated by s.

Practically, the energy stored in the hammer during its fall may be absorbed in four ways: (1) in brooming and mashing the pile either visibly at the head or invisibly at the foot or at some other part of its length; (2) in bouncing, and thus striking two or more light blows instead of one heavy one; (3) in compressing elastically the material of the pile and hammer; and (4) in causing the pile to penetrate against the resistance of the surrounding earth.

As indicated in Art. 11, brooming constitutes a serious loss of useful work whenever it occurs both directly in crushing the fibers of the wood and in cushioning the blow. Brooming at the foot does not diminish the effect of the blow on the pile head, but dissipates it more or less without useful result. It can frequently be detected by a skilled operator by a change in the behavior of hammer and pile, but not always. Bouncing of the hammer invariably means a waste of energy, either because the pile has struck a solid obstacle like a boulder,

which is soon detected, or because the hammer is too light, or the velocity is too great, or both, to get the pile in motion before it reacts elastically with more force than the hammer is exerting to push it down. A very slight rebound is a necessary accompaniment of good pile driving, due to the elasticity of the pile.

Both brooming and bouncing of the hammer cannot be provided for by any formula to determine the bearing power of a pile. When the pile is nearly home and the average penetration is to be observed for this purpose, the broomed top should be sawed off to enable a fresh surface of unbroken fibers to receive the blow. Bouncing is remedied either by providing a heavier hammer or by diminishing the fall. The elastic compression of the pile and its effect upon the bearing power were fully treated in the preceding article. In the formula now under consideration it is provided for in the margin of safety.

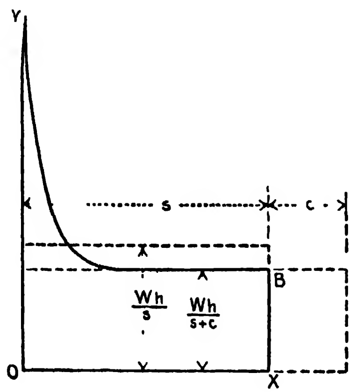


FIG. 28a.

At the instant when the hammer strikes the pile, the resistance of the pile and earth is relatively very large, but as the pile acquires velocity the resistance rapidly diminishes and then continues at some more or less uniform value until the motion of the pile ceases. The reasons for the high initial resistance are the grip of the earth upon the surface of the pile, due to its settling against it during the interval since the last blow, and the excess in the coefficient of friction at rest or at a very low velocity over that at a relatively high velocity. The approximate measure of the static bearing power of the pile immediately after driving is the comparatively uniform frictional resistance to penetration after the high initial resistance is overcome. The initial resistance is more difficult to overcome,

since the impact of the hammer occurs so suddenly and it requires time for the stress to be transmitted through the fibers of the pile. These relations are indicated graphically in Fig. 28*a*.

The initial ordinate OY represents the initial resistance and the final ordinate XB represents the final resistance as the pile comes to rest. The area of the taller rectangle represents the work done by the hammer, and its altitude is the mean resistance Wh/s. The area enclosed by the full lines has the same area Wh as the rectangle, provided no energy is wasted, and its ordinate XB represents the bearing power of the pile. The probable ultimate bearing power is accordingly equal to $Wh/(s + c)$, the term c being some empirical value to be added to the penetration. Thus, to overcome the large initial resistance is equivalent to causing some extra penetration. The value of c is doubtless as variable as the character of the ground in which piles are driven, but it was taken by WELLINGTON as 1 inch, a value which he claimed to be "based on extensive observations of the behavior of piles in driving, and on many years' experiment and study as to the general laws of friction." This value means practically that the initial resistance is about equivalent to an extra inch of penetration after the pile is set in motion. For convenience of observation, the fall is expressed in feet and denoted by H, and the penetration in inches, whence the expression becomes $12WH/(s + 1)$. A so-called factor of safety of 6 was then assumed, since it was found to give loads as large as were customary in practice based on precedent. The author made a statement four years afterward that he had discovered no cases where the factor 6 had proved insufficient, either experimentally or in service, and that, since it will in no case require piles to be driven closer together than is customary and reasonable, he should advise adhering to it in all cases unless under some very exceptional circumstances, where the engineer may see that he has special cause and justification for taking more chances. Accordingly, the Engineering News formula for pile driving with a drop-hammer is

$$\text{Safe load} = \frac{2WH}{s + 1},$$

in which W denotes the weight of the drop-hammer in pounds, H the height of fall in feet, provided the hammer falls freely, and s the average penetration in inches under the last few blows. For a discussion of the limitations of W, H and s, see Arts. 29, 30 and 31.

This formula can also be derived on the assumption that the initial resistance OY is four times the final resistance XB, and that for the first inch of penetration the resistance varies as a parabola and is constant for the remainder of the movement. Since the area enclosed by the solid lines represents the total work due to the resistance of the soil,

$$12WH = XB \times s + \left(\frac{3\overline{XB} \times 1}{3}\right)$$

or

$$XB = \frac{12WH}{(s+1)} \text{ for ultimate load.}$$

This formula was first published in the Engineering News, vol. 20, page 511, Dec. 29, 1888. It has been used more extensively in American practice than all other formulas, and at present (1925) is widely adopted as standard. The formula is modified for use with steam-hammers by substituting 0 1 in place of the constant 1 in the denominator, as explained in Art. 32.

Art. 29. Weight and Fall of Drop-Hammer

In 1897 a Committee of the American Association of Railway Superintendents of Bridges and Buildings recommended 3300 pounds as the best weight of drop-hammer for general railroad service. In hard driving, experience has frequently proved that piles can be successfully driven with a 4000-pound hammer when a 2000-pound hammer fails to do so.

In general, the hammer must be heavy enough to put the pile in motion after allowing the pile to absorb its share of the energy developed in the fall. The weight of the hammer should never be less than the weight of the pile and should preferably weigh about twice as much. In one example of piles failing to sustain without settlement half the load they were designed

to carry, the hammer used in driving was only 56 percent of the weight of the pile. In the best practice the height of fall for a drop-hammer is limited to about 20 feet. While a fall of 5 feet or even less may be used in soft ground, its value will most frequently range between 10 and 15 feet. If occasionally a fall exceeding 20 feet be used to penetrate a hard stratum, it should not be continued long on the same pile for fear of damaging it. A low fall, or short drop, has the additional advantage of securing a more rapid succession of blows, which in most kinds of earth is advantageous in securing penetration, and thus economizing time.

If W denotes the weight of the hammer in pounds, and H the height of its fall in feet, WH will represent closely the work done by the hammer in a single blow for free fall. It is considered that 30,000 foot-pounds is about as small a value for WH as it is economical to use in work of any magnitude, and that 50,000 foot-pounds should rarely be exceeded on account of the limited strength of timber.

The only drawback that may be alleged against a heavy hammer is the increased capacity required for the hoisting engine and equipment, but in work of any magnitude it is economical to provide the equipment required to do the work expeditiously and well. If the fall is too low, then nearly all of the energy developed is absorbed by the pile without producing motion; and on the other hand, if the fall is too high, its effect is analogous to that of a bullet, too large a percentage of the energy being expended in crushing the fibers in the head of the pile, or possibly damaging it elsewhere.

The resistance of the pile varies greatly from the time it is struck by the hammer until its motion stops. The mass of the pile and the higher static friction cause a high resistance at the start, while afterward the kinetic energy of the pile and the decreased friction in motion cause a much smaller resistance. On this account a heavy hammer with a low fall is more effective in securing the penetration of a pile than a light hammer with a high fall and its accompanying high velocity. In the latter case there is not sufficient time allowed to transmit

the stress through the fibers of the pile and hence too large a percentage of the energy must be expended at its head, with consequent destructive effect.

From another point of view, when the pile is ready to give out the energy received from the hammer, to produce further penetration at the foot, it is necessary to have a weight on the head to serve as an additional reaction; and a heavy hammer performs this function better than a light one.

The facts given above indicate that the height of fall should be adjusted to the resilience of the timber composing the pile as well as to some of its other qualities, including the strength in tension cross the fiber, which measures resistance to splitting. This fact was recognized practically when the committee, referred to at the beginning of this article, recommended that the fall should not exceed 12 feet for cedar piles, and 20 feet for oak piles.

It materially facilitates the progress of the work if a preliminary test is made to discover the best height of fall for the conditions existing at the given site. In one instance where it was found that the penetration was not increased by a higher fall, the average for several hundred piles observed was: fall, 14 feet; number of blows, 175; time, 20 minutes; penetration per blow, 1 inch.

When the rope is fastened to a drop-hammer and the falling hammer must pull the rope with it and thereby also revolve the drum of the hoisting engine, a considerable correction must be applied to the height of the restrained fall to reduce it to an equivalent free fall. This subject is discussed in the next article. It is the universal practice to make no allowance for the effect of wind pressure on the hammer, nor for the friction between hammer and guides when the leads are vertical and in good order.

ART. 30. THE RESTRAINED FALL

In most of the formulas for the bearing power of piles it is assumed that the drop-hammer falls freely. When the hammer is operated by keeping the line fastened to it so that the hammer

in descending must overhaul the line from the drum, it is necessary to apply a correction to the height of fall to reduce it to an equivalent free fall.

The following table gives the penetrations of three piles, driven by different pile-drivers under both conditions of free and restrained fall as reported by G. B. NICHOLSON.[1]

Pile No.	Weight of hammer, pounds	Height of fall, feet	Penetration, feet	
			Restrained fall	Free fall
1	2470	40	0 5	0 7
2	2750	45	0 7	0 9
3	2500	46	0 32	0 4

On applying the Engineering News formula for the bearing power of piles driven with a drop-hammer, it is found that the restrained falls are equivalent to the following percentages of the corresponding free falls: 74.5, 79.6 and 83.4, the average being 79.2. If the Goodrich formula is applied, in which the bearing power is inversely proportional to the penetration, provided penetrations less then ¼ and preferably less than ½ inch be excluded, the corresponding percentages are 71.4, 77.8 and 80.0, the average being 76.4.

From these data GOODRICH computed the coefficients of the final velocity of the hammer in each case to be 1.02 and 1.28, instead of 2 in the well-known formula $v^2 = 2gh$. Their average is 1.15, which agrees with that obtained by him in some experiments in which the velocity of the hammer was obtained directly, the time on the recording device being measured by a tuning-fork chronograph.[2]

Instances have been observed with new equipment, in which the conditions were unfavorable, where the required correction required for restrained fall was found to average 50 percent.

[1] See Transactions of the American Society of Civil Engineers, vol. 27, page 172, August, 1892.

[2] See the Transactions of the American Society of Civil Engineers, vol. 48, page 202, August, 1902.

When it is known how readily the resistance of the rope and the drum may be increased by the operator of the hoisting drum, and which it is difficult to guard against by inspection, it is best to disconnect the rope from the hammer and to employ nippers to secure a free fall when testing the penetration for bearing power.

ART. 31. FINAL PENETRATION PER BLOW

Formulas for the bearing power of piles are not designed for the case where a pile is driven through soft, yielding material, like silt and wet clay, to a hard stratum of sand, gravel, hard-pan or rock, for then it acts like a column and must be designed accordingly. Column action also exists where a pile is driven through strata of varying consistency and the lower part of the pile penetrates a stratum which is more compact than those which lie above it. In this case the lower end may be almost completely restrained, while the upper end may be restrained but slightly.

Formulas for bearing power are intended primarily for the general case in which the support of a pile is due to frictional resistance between the surface of the pile and the surrounding earth. Extended experience has proved, however, that their use may properly be extended to a pile which receives some additional resistance in bearing at its foot.

The value of the penetration to use in the formula is generally taken as the mean for the last five or ten blows. When the drop-hammer is employed, no value less than ¼ inch should be considered in any case for timber piles, and usually not less than ½ inch for hardwood piles, nor less than 1 inch for softwood piles. When the penetration is smaller than the values just given it is highly probable that the true penetration of the foot of the pile is not equal to the movement of the head of the pile where the measurement must necessarily be made.

Even the average penetration under the last five blows is not a fair measure of the bearing power of the pile at that

time, unless the penetration has been either uniform for a number of blows, or decreasing at an approximately uniform rate, and that it would continue in the same manner for a short distance farther. The last condition mentioned should be known from previous explorations of the ground. It is presupposed that the head of the pile is sound, that the weight of the hammer and height of fall conform to the limitations indicated in Art. 29 and that the operation of driving is not interrupted materially.

In practice, a penetration of less than ½ inch should be regarded with far more suspicion than is frequently accorded to it. An apparent penetration below this limit is more likely to be merely a sinking of the head due to crushing the foot or crippling the pile in some other part of its length.

In hard driving, experienced inspectors are often misled when they do not know the character of the different strata from previous explorations. Whenever the observed penetrations are quite irregular, caution is especially necessary. It is by no means difficult to get an apparent penetration with the foot on solid rock for a spruce pile under a 2000-pound drop-hammer by continued driving. Damage due to overdriving is discussed at length in Art. 19.

The value of the final penetration per blow depends upon the nature of the ground, the size of the pile, the smoothness of its surface, its taper, the form and diameter of the tip, and the total penetration, as well as upon the weight and fall of the hammer and some other minor factors, but extensive tests have shown that the final penetration under a given energy of the hammer is practically as good in determining the bearing power of the pile as a test by actual loading.

Whenever a follower has to be used on top of a pile as it is driven home and it is desired to compute its bearing power, it is necessary to apply a correction to the observed average penetration. For this purpose some tests should be made under fairly comparable conditions when each pile may be driven alternately with and without the follower.

ART. 32. FORMULA FOR STEAM-HAMMER

It is observed that the dynamic effect of the short, quick blows of a steam-hammer exceeds that of the slower blows and higher falls of a drop-hammer more than a mere comparison of the relative energy developed seems to warrant. This is readily seen if a 3000-pound drop-hammer is operated with a fall equal to that of a steam-hammer having the same weight of moving parts. The greater efficiency of the steam-hammer is due primarily to the rapidity of its blows, which does not permit the material penetrated to settle back against the pile after being shaken up, or pushed away. As indicated in Art. 34, experience shows that 12 to 24 hours' rest requires the number of blows per foot of penetration to be increased from 6- to 10-fold. Another effect of rapid blows is that before the pile comes fully to rest the next blow is delivered, hence the variation in resistance is not so marked as that illustrated in Fig. 28*a*. Accordingly, the value of the constant in the denominator of the Engineering News formula is dependent chiefly upon the magnitude of the time interval between blows. When, therefore, the method of pile driving is so radically changed as by substituting a steam-hammer for a drop-hammer, the constant must be reduced materially. WELLINGTON adopted the value of 0.1. The Engineering News formula for pile driving with a steam-hammer is, therefore,

$$\text{Safe load} = \frac{2WH}{s + 0.1},$$

the significations of the terms being the same as those given in the sixth paragraph of Art. 28.

At the Brooklyn Navy Yard two test piles were driven under probably as nearly equal conditions of the ground penetrated as may be possible in practice. One was driven by a steam-hammer with moving parts of 3 tons and a stroke of 3 feet, and the penetration per blow decreased steadily from 4 inches to ½ inch. The pile was 20 and 14 inches in diameter at the butt and tip respectively and was driven to a total penetration of 43 feet in 7 minutes by 373 blows. The other was driven

by a 1-ton drop-hammer which started with a fall of ½ foot and gradually increased to 35 feet as the pile went down in the leads. It was driven to a total penetration of 45 feet with 735 blows in 166 minutes. The final penetration was 1¼ inches. This pile had an iron shoe, while the one driven by steam-hammer had none. According to the Engineering News formulas the corresponding safe loads are 30 and 31.1 tons. Another pile with a penetration of 33 feet driven by the drop-hammer with a fall of 30 feet developed an ultimate resistance of 125 tons. Such close agreement is, however, by no means common between the results of driving by both types of hammer since the effect of rest on the bearing power varies in different material and between slow and rapid driving.

Some tests made during the foundation work for the Pearl Harbor dry dock at Hawaii and the Mare Island dry dock indicated that the constant 0.3 gave results that compared better with the drop-hammer values than 0.1. Similar results were obtained on tests made by the engineering department of the Norfolk and Western Railroad.

For the double-acting hammer the formula

$$\text{Safe load} = \frac{2H(W + AM - b)}{(s + k)}$$

has been proposed, where A denotes the effective area of the piston, m the mean effective pressure, b the total back pressure, k a constant and the other terms as given above. Since the rapidity of blows is greater than for the single-acting hammer, it is suggested that the value of k be smaller. If for the single-acting hammer delivering 60 blows per minute k is taken as 0.3, then for the double-acting hammer delivering 100 blows per minute the value of k should be 0.18.

The Navy Department has used the above formula, making the value of $k = 0.3w/W$, where w denotes the weight of the pile in pounds and W, the same as above, the weight of the ram of the hammer. This is a logical type of formula, for, as explained in Art. 53, the relation between weight of pile and that of hammer must be considered in any rational pile-driving formula.

ART. 33. TABLES AND DIAGRAMS

A convenient table for the safe load on piles, for a given weight of drop-hammer, may be prepared by placing the fall in feet at the top of a column, and the penetration in inches at the left end of a line. The falls may include each foot between suitable limits, while the penetrations begin with ¼ inch and vary by quarter inches at first and then by half inches, to the desired limit. The loads may be expressed in units of 1 kip = 1000 pounds, or in tons, as preferred, and then inserted in the table within practical limits.

In practice, diagrams have been found more convenient than tables and a number of different forms have been devised. In one diagram the fall is laid off as an abscissa, and the safe load as an ordinate, the value of each penetration being marked on a line connecting the proper points of intersection of vertical and horizontal coordinates. For the Engineering News formula, or GOODRICH'S formula, these lines are straight.

A very simple one for use with a given weight of hammer and a standard height of fall may be constructed by laying off the average penetration as an abscissa and the safe load as an ordinate. A curve is then drawn connecting the proper points of intersection of horizontal and vertical coordinates. The diagram shows at a glance what penetration is required for a given safe load, or *vice versa*. For a steam-hammer the penetration is preferably expressed by the number of blows per inch. When driving test piles for foundations on the New York Barge Canal rectangular diagrams were used based on the Engineering News formulas for drop- and steam-hammers respectively. On the first one the fall of the hammer is laid off along the bottom from zero at the right end to 50 feet at the left. The safe load is laid off along the top from zero at the left end to 100,000 pounds at the right. The values of $2WH$ are laid off on the left side from zero at the bottom to 168,000 foot-pounds at the top. A series of lines radiating from the lower right corner are drawn for weights of hammer from 1000 to 5200 pounds. Another series of lines radiating from the lower left corner are drawn for pene-

trations from ¼ inch to 3 inches. The diagram is used by following the diagonal line for weight of hammer to its intersection with the vertical line for the height of fall; from this point a horizontal line is followed to its intersection with the diagonal line for the penetration; on the top of the vertical line through this point the safe load is stated.

The second diagram for use with steam-hammers is constructed on the same principles. The fall of the hammer

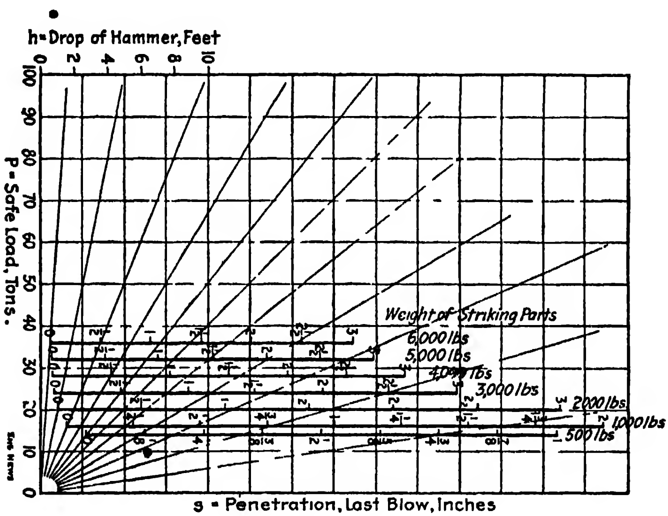


FIG. 33*a*.

extends to 5 feet, the safe load to 100,000 pounds and the values of $2WH$ to 45,000 foot-pounds. Four weights of striking parts are given—550, 1800, 3000 and 5000 pounds— while the penetrations include ⅛ inch in addition to those inserted in the first diagram.[1]

Figure 33*a* illustrates an ingenious diagram, devised by A. S. MILINOWSKI, which is based on the Engineering News formula for steam-hammers. A line connecting the point Q with the intersection of the coordinates h and P passes through the corresponding penetration. On a full-size diagram, the diagonal lines are replaced by a thread fastened at the point Q.

[1] See Engineering Record, vol 56, pages 720 and 721, Dec 28, 1907.

A diagram which contains no radiating lines, and is very easy to read with precision, may be constructed on logarithmic paper, as shown in Fig. 33*b*. The vertical coordinates give the total penetration in inches for the last 10 blows of the steam-hammer, while the horizontal coordinates give the safe loads. A similar diagram for a drop-hammer with falls of 20 feet and less is given in an article by E. F. KRIEGSMAN in the Engineering Record, vol. 65, page 417, April 13, 1912.

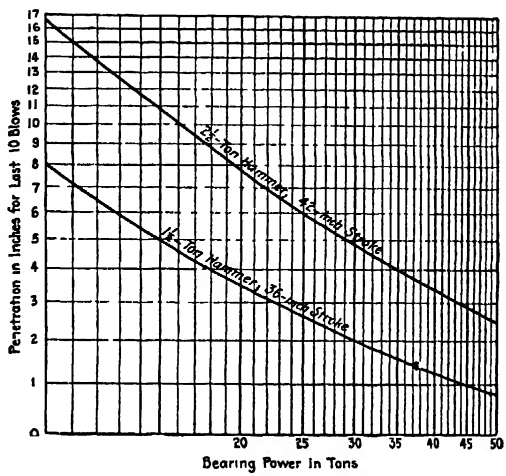


FIG. 33*b*.

ART. 34 EFFECT OF REST ON BEARING POWER

In some kinds of material, like sand or gravel, if a pile be partly driven one day and driving is resumed the next day, the resistance as measured by the average penetration per blow at the beginning of the second day's driving is generally found to be practically the same as that at the close of the first day. This is not the case with other kinds of material, for which increases in resistance within 24 hours of more than 1000 percent have been observed in extreme cases.

An old contractor reports an illustration of this phenomenon: An attempt was made to reach hard bottom through a very deep marsh. After driving a 35-foot pile another one of equal length was spliced to it and also driven without finding a hard stratum, sinking 4 or 5 inches per blow. After driving the next pile to a penetration of 25 feet it was time to quit work, hence the pile was left in the leads in that position. The next morning five or six blows of the hammer failed to produce any appreciable movement and accordingly the engineer concluded to drive piles about 35 or 40 feet long and to depend upon the friction in the soft meadow muck to support them. The trestle bridge thus supported carried its traffic safely for years, the locomotives and cars becoming much heavier than it was originally expected would ever be used upon it.

On the 6-mile pile trestle bridge of the New Orleans and Northeastern Railroad crossing Lake Pontchartrain the piles were from 45 to 70 feet long. The longer piles often penetrated 5 feet under their own weight, 3 feet more when the hammer rested on top, 20 feet additional at the rate of about 2 feet per blow for low drops and, finally, from 10 to 15 feet deeper with a penetration of about ½ to 1 foot for a 3000-pound hammer dropping 10 feet. The driving was done rapidly by means of a friction drum. It was observed that if a pile was allowed to stand several hours owing to a breakdown or some other cause, several blows were required to start it, but later the penetration resumed the same rate as before the period of rest. The engineer in repeating the facts 15 years later stated that none of the piles had settled under the traffic.

At Fort Point Channel, Boston, borings 65 feet below low water showed only soft blue clay changing in a few instances to soft yellow clay. Tests were made to determine the increased frictional resistance after a period of rest. A spruce pile 35 feet long, 17 inches at the butt and 7 inches at the tip was driven 20 feet into the clay by a 2360-pound hammer falling 8 feet. The average penetration for the last five blows was 5.5 inches. After four days' rest, the pile was struck 20 blows, giving an average penetration of 0.9 inch for the first five blows and 1.5

inches for the 20 blows. With another pile in slightly softer material the average penetration for five blows before and after four days rest decreased from 7.6 to 1.6 inches.

In the upper San Francisco Bay is a deposit of very soft silt due to the finest and lightest tailings from hydraulic mining in the Sacramento and San Joaquin rivers. A pile sinks 20 to 27 feet in it by its own weight, but with a total penetration of 40 to 55 feet will later support 40,000 pounds. If a pile is allowed to rest only 15 minutes, a heavy blow from a drop-hammer will not move it perceptibly, but a few blows in rapid succession will start it at the old rate. While attempting to splice a pile on one occasion the mud settled against it so that it could not be driven further and a scow with a displacement of 30 tons could not lift it. The friction developed was 200 pounds per square foot.

These examples show why it is possible to sink piles in soft ground by a static load placed upon them which afterward will safely support a load several times as great. Instead of superimposing a static load, the pile may be pulled down by means of a block and tackle operated from a scow. It is also instructive to reflect upon the fact that a pile driven to a penetration of 90 feet in 10 minutes with apparently small resistance, a few days later supported a test load of 40 tons, whereas a pile of this length could not support it without lateral support if its foot stood on solid rock.

A similar decrease in penetration after rest overnight has often been observed in ground where the average penetration per blow was less than an inch. In one instance the penetration was reduced in the ratio of 2.3 to 1 and in another from 2.8 to 1. At the Brooklyn anchorage of the Manhattan bridge it was noticed frequently that after a number of days' rest the resistance of piles would develop to meet the requirements, although the driving was regarded to be hard.

The increase in supporting power of plastic muds or clays is due to the material settling back against the surface of the pile which was disturbed in driving, the vibration of the pile temporarily enlarging the hole and thus releasing part of the

frictional resistance. This curious physical property is analogous to that of India rubber. If a pin be forced into a solid India-rubber ball, the same force which pushed it in can pull it out again, provided it be done immediately, but after waiting 24 hours the force required will be about five times as great. A similar effect is produced in sand which for certain properties of moisture will temporarily arch itself laterally over small areas so that the pile will not receive pressure until some time after driving.

ART. 35. EFFECT OF SUBSURFACE CONDITIONS

When a pile is held in position entirely by frictional resistance the load is transferred to the adjacent ground and transmitted down to different levels in widening areas until a level is reached where the earth can readily support the unit bearing value. The mass of ground thus transmitting the load may be said to form approximately a conoid of pressure, the slope of which depends upon the nature of the ground with respect to both the kind of material and its degree of compactness. When the coefficient of frictional resistance is small, the total penetration must be larger or the pile will settle under the load, sometimes slipping through the surrounding material and at other times carrying a mass of the earth with it.

When the bearing power of the earth chiefly supports the pile at its foot, it should be designed as a column, as described in Art. 26. When the support is due mainly to friction, an important criterion of its magnitude is the final penetration per blow of the hammer; but in first-class practice this is supplemented by a knowledge of the characteristics of the ground at the site. It is important to know, for instance, whether the resistance will increase for some time to a maximum, whether it will remain nearly or quite the same, or whether there is any possibility of its diminishing.

An instructive example is that of six yellow pine piles driven into the wet alluvium of New Orleans which supported a turntable. Their diameters were 14 and 12 inches at the butt and tip respectively, and the average length was 31 feet below

the cut-off. When driven by a 2825-pound hammer falling from 30 to 35 feet, penetrations 9.5 to 18 inches were obtained under the last blow, or an average of 12.1 inches. The total number of blows for each pile averaged 27, showing that there was practically no appreciable compacting of the ground. The weight of the turntable and the two courses of creosoted timber grillage to which it was bolted was 36,100 pounds, and the weight of engines and tenders ranged from 116,000 to 156,000 pounds. No settlement had been observed during the nine years of operation after its construction, when the case was reported.

In another instance piles 60 to 70 feet long were driven into mud with a final penetration of 2 feet or more per blow, but examinations for a number of years failed to give any evidence of settlement under traffic. These piles were used in some bridge foundations on the Boston and Maine Railroad near Conway Junction, Mass.

It is particularly important to know what the effect of time is upon the supporting power of piles when the penetration per blow is very large. In Art. 34, reference was made to the very fine silt in upper San Francisco Bay. Even in that extreme case the friction was found to be about 200 pounds per square foot. The piles supporting the turntable in New Orleans cited above developed a frictional resistance of about 300 pounds per square foot under a load which was certainly safe. Whenever crusts of vegetable or of peat-like deposits cover sections of swamps which may be deep and strong enough to support ordinary highway loads it is usually necessary to penetrate them for pile foundations and extend the piles to the sand or clay bottom.

Piles have been driven through 10 to 18 feet of soft mud and 15 feet into soft clay, when it took three or four blows of a 2000-pound hammer falling 6 to 12 feet to secure a penetration of 1 foot. Immediately after driving, the piles might be swayed as much as 2 feet at the head which was 15 to 20 feet above the mud line, but at the end of a week it required considerable force to move them. In attempting to pull piles which penetrated only 5 or 6 feet into the clay they frequently broke off.

The effect of sand upon the settlement of piles having insufficient length, but for which the penetration per blow was very small, is described in Art. 36. By means of a steam-hammer aided by a water-jet, piles which have been driven 33 feet into quicksand, and apparently might have been driven twice that depth, could not have been driven to a quarter of that depth by a drop-hammer alone. In the upper drainage district of New Orleans a sand stratum underlies the silt at a depth of 40 feet. In driving piles they "bring up" almost as suddenly as if they struck solid rock. The difficulty of driving piles in gravel increases in proportion to its fineness, if the ordinary drop-hammer method be employed.

Unless sand or gravel are mixed with other material they are practically incompressible and have to be displaced in driving piles. On account of the danger of scour it is often necessary to secure a large total penetration, and in such cases it is unnecessary to consider the final penetration per blow. The strength of the material of which the pile is composed limits its bearing power.

Engineers who have closely studied hardpan and sandy clays claim that in this material the most perplexing results of all are to be found. No two hardpans seem to develop the same results. The ultimate result depends not only upon the percentage of clay and sand in the hardpan, but also upon the solubility of the clay when brought into contact with the local ground water. Where piles are driven through ground water overlying hardpan, the water invariably follows each pile down as it penetrates the ground, softens the clay in contact with the surface of the pile and often practically destroys all lateral friction. In such cases the only point of support is at the end of the pile, and the overhead load will be supported by a cluster of columns, the heights of which are equal to the lengths of the piles.

ART. 36. ON TOTAL PENETRATION

It is well known from observations in practice that piles driven in sand will sometimes settle when set in vibration by

a live load. For example, in a pile trestle bridge, piles have settled under a load of 9 tons each, although they had been driven to an average penetration of ½ inch with a 1200-pound drop-hammer falling 20 feet. The final penetration is no adequate index of the bearing power of the pile in this case, since the hammer was too light in proportion to the pile, but, apart from that, the primary reason for its settlement was its lack of sufficient depth of total penetration to prevent the vibration from being communicated to the foot of the pile. In another instance piles which were supposed to have been driven to absolute refusal, settled 15 inches under a load of 19 tons each.

Observations seem to show that this tendency for piles to settle in sand is independent of the penetration under the last blow. Where the track is at a considerable elevation above the ground the specified final penetration per blow is reached when the total penetration is so small as to permit the pile to rock slightly on its foot. The effective remedy for such conditions is to drive the piles deeper. This can readily be done without injury to the pile since, fortunately, sand is so well adapted to the use of the water-jet. A total penetration of 10 feet in sand may be sufficient for piles in a building foundation, but may be insufficient for a pile trestle.

It is noticed that if the longitudinal reinforcing bars of a concrete pile are hit by the hammer instead of being protected by either plain concrete or independently reinforced concrete above it, that the foot of the pile is far more liable to be injured in driving. The steel seems to transmit vibrations to the foot which would be dissipated otherwise before reaching it. The injury to the foot of the concrete pile implies more vibration at that point and therefore makes it somewhat analogous to the timber pile which settles by the lateral movement of its foot.

All the piles under a building should be driven to the same depth if possible and the areas of groups should be carefully proportioned to the loads to be supported unless the spacing is large enough for each pile to develop its full supporting power independently. If the earth is not uniform in character, the

piles should be driven preferably through the variable stratum to one which is practically uniform.

When driving piles for the foundation of a cylinder pier it may not be possible to secure the same depth for all the piles in any cylinder on account of the gradually increasing compression of the ground unless the piles first driven are not required to have as small a penetration per blow as that specified. In one example, where the average penetration of all the piles in a cylinder 20 feet in diameter was 31 feet, the total penetration for the last five piles driven in the cylinder averaged 24 percent less than the penetration for the first five piles.

The following sentence occurs in COOPER's General Specifications for Foundations and Substructures of [Country] Highway and Electric Railway Bridges 1902: "The minimum penetration accepted for the piles should be about 8 to 12 feet in wet gravel, sand, or stiff clay, and 20 to 40 feet in soft clay or silt." These values are apparently intended for foundations of substructures supporting light superstructures.

Whenever pile foundations are liable to scour during exceptional floods, provision must be made for this by increasing the total penetration beyond the depth needed to secure the specified average penetration per blow.

ART. 37. DEGREE OF SECURITY

The force F in equations (1) and (2) of Art. 27 represents the final pressure on the head of the pile as it comes to rest after any blow of the hammer, but since an average value of the final penetration is used for a certain number of blows, it is assumed to equal the ultimate bearing power immediately after driving. This bearing power is liable to change in most cases for earth of different kinds. Generally, it increases after driving up to a maximum, and this maximum does not increase with time. In Art. 34 a number of examples were cited to show how large an increase may occur in short intervals of time, but, on the other hand, a change in the moisture conditions due to hydraulic constructions may transform a stratum of clay into a slowly

yielding mass which permits piles to sink into it that before appeared to have a solid support, or deformation in the ground of a contiguous site may reduce the bearing power in a given site. When a pile acts as a column, the stress in the outer fiber due to its safe load must be less than the elastic limit of the material under a long-time test irrespective of the relation between the elastic limit and the ultimate strength of the material for direct compression in an ordinary test of a short specimen.

The safe load for a pile in which the resistance depends upon friction only is analogous to that of a column and must be less than a load which will cause settlement under a long-time test. In some instances it may be difficult if not impossible to determine how much of the supporting power is due to bearing on a solid substratum and how much to friction alone. In others there is no guarantee that a pile will not steadily sink under a heavy quiescent load applied continuously, although it withstands satisfactorily the specified test of driving. This result is especially to be feared in clays.

Experience in testing piles by static loads placed upon them shows that a load, which produced no settlement when first applied or even during the first day, may cause settlement if left on for several days or a week, or a load may cause increasing settlement for a time but after a while no further settlement will take place. What margin of security is to be allowed between this load and the safe load for which the pile is to be designed depends on a number of factors.

Sometimes the load to be supported by piles is a dead load, while in other cases it is a live load. The live load itself may increase during the life of the foundation as, for example, that of locomotives and trains passing over a railroad structure. In pile trestles it is none too great an allowance to assume that the entire weight on the driving-wheel base falls upon each bent in succession In addition to the static weight of the live load, some provision must often be made for the dynamic effect due to a moving load or to the vibration of machinery. At a factory on Fort Point Channel, Boston, the movement of piles may be observed when the machinery is in motion. The allowance thus

made for foundations is usually less than for the superstructures or for parts of the substructure. If a building is adjacent to a railroad, some account must be taken of the fact in designing its pile foundation. In some structures, like that of a wharf, the failure of piles may cause serious loss of property or even loss of life, and hence a larger margin of security is needed. In other cases it is difficult to estimate the probable load. A higher load may often be used for piles under temporary structures than that allowed for permanent structures.

A building may be subsequently used for a different purpose than that for which it was designed. The building itself may be readily strengthened on this account, but it may be impracticable to increase the strength of the foundation without excessive cost. Such contingencies are provided for only in special cases, for it is not generally economical to make the additional investment required. It makes a decided difference whether the structure to be supported is permanent or merely temporary.

The most important factor is that relating to the nature of the ground which is penetrated by the piles. The more uncertainty which exists in regard to it the larger the margin of security must be. The character of the earth also determines whether the bearing power of all the piles in a foundation, or in some portion of it, equals the bearing power of one pile multiplied by the number of piles. Extra caution should be used when the penetrations of piles are quite variable.

Some engineers after evaluating these several factors in a given case, determine the safe load with reference to the ultimate strength as obtained by a formula for the bearing power, while others fix it with reference to the value obtained from a formula that has been divided by a factor to make it certainly safe for the most unfavorable conditions met in ordinary practice. The latter hold the opinion that it is better in principle to be obliged to consider carefully whether the conditions relating to the foundation have been so fully investigated as to justify any increase in the working load per pile for adequate security, rather than to start with a value which is certainly unsafe and reduce that, thus enabling a man to deceive himself

with the notion that he is cautious, when he is really rash. These conflicting opinions lose their force when adequate tests have been made to determine the character and the behavior of the earth to be penetrated.

Uncertainty with respect to the load to be supported by the foundation is best provided for by the addition of some estimated percentage, after due consideration of all the facts and probabilities.

The statement is frequently made in engineering literature that pile driving is largely a matter of judgment and that theoretical considerations have practically no part in it. It should be remembered, however, that foundation failures and lack of economical design are most frequently due to a failure to explore the subsurface conditions. Under this condition, the so-called exercise of engineering judgment is practically equivalent to guessing.

It is interesting to note the results of this method as indicated in engineering periodicals: "It is an established custom among engineers to restrict the loading of timber piles within the range from 6 to 12 tons." "I note that there is a pretty general working rule among engineers throughout this country to allow a load of 20 tons each on all piles driven to practical refusal."

The art of pile driving at its best is based on science. Scientific method consists mainly in the solution of a problem by analysis into its component parts and by treating each part separately. With the diminishing supply of good timber piles and their increasing cost, the methods of design for pile foundations should have approximately the degree of precision which is applied to the design of the superstructure.

The margin of security is well illustrated in the following record regarding timber piles driven in 1908 to support the falsework of bridge No. 5 of the Norfolk and Western Railroad over Elizabeth River at Norfolk, Va. "The falsework piles were driven with a 3300-pound hammer falling 10 feet. They were 70 feet long and had a penetration of about 40 feet. The average penetration at the last blow was 2 inches and by the Engineering News formula the safe load is 11 tons. These piles,

however, safely carried 24 tons per pile and it required a pull of from 30 to 35 tons to pull them out. The river bottom is composed of about 10 feet of soft river silt overlying stiff blue mud and sand in layers of varied thickness"; this formation extending to a depth of over 1500 feet.

When WELLINGTON introduced the factor of 6 in his formula to obtain the safe load, he practically assumed that no previous exploration of the ground is ordinarily made. He stated also that "the average factor may be taken as somewhat under 4." Some consulting bridge engineers recommend 3, and others 2.5, it being remembered that the factor will probably be doubled with the lapse of time in many kinds of earth and with a possibility in some cases of increasing it 4-fold.

It is interesting in this connection to note that the ultimate load by GOODRICH's formula is 2 times the safe load by the Engineering News formula when the average penetration is 5 inches, 2.5 times for a penetration of 2 inches, 3.3 times for a penetration of 1 inch, and 5 times for a penetration of ½ inch.

ART. 38. TEST PILES

The method of computing the bearing power of a pile by means of the observed final penetration per blow due to a hammer having a given weight and fall is described in previous articles of this chapter. The practical limitations of the terms included in the formulas are also discussed, so that the results obtained may conform reasonably to the practical conditions of pile driving. Since in most cases the ground to be penetrated is not homogeneous throughout the site, yet the conditions do not differ so much as to make it necessary to determine the bearing power of every pile nor even for a large percentage of them. Piles driven to test the resistance of the ground, or to determine what length of pile is required to support a specified safe load, are called test piles.

In order to secure uniformity in conducting such tests for the foundations of structures on the New York Barge Canal, instructions were prepared accompanied by suitable diagrams

(see Art. 33) to economize time. The required loads were given on the plans and in the specifications. The instructions contained the following items: Test piles are to be driven in each foundation after it has been fully excavated. One test pile is to be driven for each 500 piles, or less, in the structure, but not less than three piles of each class, for each structure, and they shall be located so as to develop the conditions over the entire area. If the test is to be made with a steam-hammer, the weight of its moving parts and the effective stroke of the hammer are to be ascertained. A pile is to be selected somewhat longer than that probably required and driven in the proper location. When the penetration under each blow becomes approximately uniform and nearly equal to the value required, the hammer is stopped and its position marked on the leads. After striking 10 blows, its position is again marked, and the penetration under the last blow is taken to be the average penetration for the 10 blows. The safe load is then to be computed by the Engineering News formula, or obtained from one of the diagrams. This process is to be repeated, if necessary, until the pile is driven to a depth that gives the required supporting power. After measuring the length extending above the plane of cut-off, and deducting this from the original length of the pile, the required length is obtained.

If the test is made with a drop-hammer the elevation of its bottom is marked similarly before and after the test blows are struck. The engineman is directed to raise the hammer to about the same height (15 to 20 feet) each time before letting it fall. An observer standing directly in front of the leads is to note the height reached by the hammer before each blow by means of a scale of feet painted on the leads. The average fall of the hammer to be taken as the difference between the mean of the elevations of the two pencil marks and the average height reached by the hammer. The penetration for the 10 blows is obtained by careful measurement as before.

Some specifications require the corresponding average for only the last five blows to be taken. Sometimes engineers require a "trip" to be used in driving test piles with a drop-

hammer, in order to insure a free fall. The only objection to this practice is that the driving is much slower than that to be used for the regular piles. In many cases it is necessary to know the length of piles required before the site is excavated. Test pits may then be dug to the elevation of cut-off, and piles driven in them with the aid of extension leads. As indicated in Art. 31, penetrations less than ¼ inch for a drop-hammer should be excluded, and it is desirable never to use a hammer which is lighter than the pile. The head of the pile must be sound and not in the least crushed. The general observations made during the progress of regular pile driving later will indicate whether an additional test pile may be needed in any part of the foundation site. Certain engineers drive test piles some distance below the depth required to support the specified load in order to test the stratum below the foot of the regular piles and thus ascertain the conditions more thoroughly.

Before building the trestle approaches of the Dumbarton bridge at San Franciso Bay, test piles were driven at intervals of 300 feet along the proposed bridge axis. Under the mud from 2 to 18 feet deep was found a stratum of fine black sand mixed with gravel and 15 to 20 feet deep, which, in turn, was underlaid by a bed of hard clay. It is interesting to note that no trace of clay was found by the Spring Valley Water Co. when making tests only about 500 feet to the north on a line parallel to the Dumbarton structure, thus indicating that it is not wise to fail to make tests on a site because the results of other tests in the vicinity are known. When guide piles are used in soft material, their lateral resistance should be tested to determine the length required on that account. This is unnecessary if the guide piles are practically relieved from bending as cantilevers by horizontal or diagonal bracing.

The final test of the bearing power of a pile consists in subjecting it to a static load. Two methods have been used in applying the load; in one the load is balanced on a single pile, and in the other the load is placed on a platform supported by three or four piles. The latter takes more time and involves greater expense for hauling and handling pig iron, brick or other

loading material. It requires no more trouble to balance the load on one pile than to guard against the unequal settlement of several piles. Sometimes it may be more economical to apply the load by means of a heavy timber lever, the short end of which is anchored to several other piles. A leverage of 3 or 4 to 1 may easily be obtained, and cement in bags may be used as loading material.

In case the piles in a foundation are expected to act as columns, the results of loading test piles should not be depended upon unless they are sufficient in number to insure their action in a similar manner, and they are stayed against lateral motion. Since loading tests in soft material show that a single heavily loaded pile may carry down with it other unloaded piles driven at considerable distances away, it proves that the surrounding earth lacks resistance rather than the piles, and is therefore being tested. Where dependence for the supporting power of a pile is placed upon a deep-lying firm stratum, both theory and experience have shown that the depth of the stratum below the surface has an appreciable effect on the actual supporting power afforded the bottom of the pile. In such a case E. P. GOODRICH recommends the sinking of a large pipe and after the soil has resumed its natural condition, to insert through the pipe a heavy timber with a blunt cap just large enough to pass through to the earth to be tested. A platform on top of the timber can then be loaded and accurate observations made to determine the compressibility of the ground in amount and rate under different loads. The latter should be alternately increased and decreased to learn whether any elasticity exists, and the experiment should be continued over several days or weeks, in order to ascertain whether the earth under that test possesses any characteristics comparable with the viscosity of certain materials. Pipes and plungers of several sizes should also be employed. The pipe should be sunk by dry methods and accurate data secured of the weight per cubic foot, amount of humidity and internal friction coefficient.[1]

[1] See Proceedings of the American Railway Engineering Association, vol. 11, page 217, 1910.

In loading test piles the weight should be applied in increments after its magnitude is about two-thirds of the computed safe load. An interval of at least one day should be allowed between the application of the increments of load in order that the greatest load may be known which just fails to produce continued settlement after the initial settlement for that load has occurred. In material like pure sand or gravel it is unnecessary to apply test loads, since its supporting power is greater than that of the compressive strength of the pile itself.

As a period of rest increases the resistance of a pile in soft material as described in Art. 34, it seems reasonable to compare the bearing power computed from the penetration after rest with the greatest static load that can be placed upon the pile without increase in settlement for at least 24 hours. In the third paragraph of Art. 34 data are given relating to six piles which carried a load of over 25,000 pounds each for a number of years, but if the safe load is computed by the Engineering News formula from the penetration obtained before a period of rest, it is found to be only 13,000 pounds. In another example a test pile 92 feet long, 16 and 8 inches in diameter at the butt and tip, was driven 73 feet into the mud of San Francisco Harbor, the penetration being 3 inches under the last blow of a 2900-pound hammer falling 20 feet. The computed safe load is 29,000 pounds. The pile carried 90,000 pounds of pig iron on a platform built around the pile, and no further settlement was perceptible after an interval of 24 hours. Such a load may be properly regarded as the elastic limit of the pile's strength.

In New York Harbor a pile with an average diameter of 12.5 inches and 82 feet long was driven through 30 feet of muck and 33 feet into loam and sand, by 24 blows of a 3800-pound drop-hammer with fall of 10 feet, the penetration under the last blow being 9 inches. The computed safe load is only 7600 pounds. Six days later a static load of 20,000 pounds (the specified load to be carried) was placed on the pile and caused no settlement. It was tested again 39 days after driving by a 5000-pound drop-hammer falling 15 feet, and the penetration under the first blow was found to be 1 inch.

The ground formation at Atlantic City is clean sand interlaid with thin beds of clayey sand to a depth of 400 feet. Three sets of tests were made on groups of four piles each, 30-foot piles being used for the first two tests and 15-foot piles for the third. In the first test the piles, averaging 8 inches in diameter at the point and $11\frac{1}{2}$ inches at the butt, were water-jetted into place, a hammer being used only for the final sinking. The maximum pile load was 95,000 pounds, or 1220 pounds per square foot if the load was carried entirely by friction. At a pile load of 59,000 pounds the average settlement was 1.06 inches, which increased to 2.78 inches on the application of full load. Three days later it was 3.28 inches, with no increase at later dates.

The second set of tests differed from the first only in that no hammering was done, the entire sinking being effected by the water jet. The maximum load per pile was 86,000 pounds and the average settlement 4.3 inches.

The piles of the third group averaged $6\frac{1}{2}$ inches in diameter at the point and $8\frac{5}{8}$ inches at the butt, the penetration being 15 feet. These piles were driven in the same manner as the first group. The last three blows of a 1200-pound hammer falling 10 feet caused no penetration. This pile group failed at an average load of 72,000 pounds by the piles sinking until the loading box rested on the surface of the ground. At 50,000 pounds the settlement was 0.53 inches and at 68,000 pounds 1.42 inches. The average unit friction at failure was 2400 pounds per square foot.

Sometimes the driving of test piles does not give sufficient information about the proper location of the foot of a pile. For example, a given stratum of gravel underlaid by soft material may be thick enough to carry the load by bearing the piles on its surface, but will not do so if the piles penetrate it for some distance. An instructive example of this was quoted by WELLINGTON in Engineering News, vol. 22, page 368, Oct. 19, 1889. In such cases and where test piles are found to be too expensive on account of moving a pile-driver to the site, a very careful investigation should be made of the subterranean strata by means of test borings in several parts of the proposed

site; good wash borings will generally supply this information, but core borings are more reliable, as well as more expensive. In certain kinds of earth, borings made by an earth auger are satisfactory and less expensive (see Art. 178). The borings will show the depths of the various strata in different parts of the site, the location of ground-water level, and the stratum which will afford the necessary support. In certain special conditions of the ground, however, it may be insufficient to have either test piles or borings alone, both being necessary to determine the conditions adequately.

Art. 39. Pile Records and Performance

The number of timber piles which can be driven by one pile-driver gang in a day depends upon many factors. The size of the pile; the depth to which it is driven; the kind of ground which is penetrated; the degree of moisture which it contains; the kind of hammer used, whether a steam- or drop-hammer; the relation of the weight of hammer to that of the pile; whether a cap or a ring is used to protect the pile; whether a water-jet is employed or not; the training and experience of the crew; the character and condition of the pile-driver and its equipment - all have their effect upon the result obtained. Frequently, too much time is lost in those operations which do not involve the action of the hammer, like moving the pile-driver from one position to another, getting the pile to the driver, placing it in the leads, etc.

A pile-driving record was established in 1918 when a 12-man crew at Hog Island drove 220 piles averaging 65 feet in length in 9 hours and 5 minutes. The equipment consisted of a Vulcan No. 1 hammer and a skidding rolling machine, with a three-drum 9 by 10 hoisting engine, both hoist and hammer being driven by compressed air. On the foundations of the Cambridge bridge over the Charles River built in 1901, the average day's work for a crew was considerably over 100 piles per day of 10 hours, while the highest number driven in a single day of nine hours was 212. The piles were about 40 feet long,

and the heads were driven with the aid of a follower to an elevation of 18 feet below low water. The material penetrated was a hard clay below the upper stratum of softer material. The moving parts of the steam-hammer employed weighted 5000 pounds.

As an example of the relation between the average, the minimum and the maximum, the performance at the Mare Island Dry Dock No. 2 may be cited. The average number of piles driven per shift of eight hours was 35 for a period of three months, 74 piles representing the best day's work. The timber piles ranged from 40 to 65 feet in length, and the penetration varied from 12 to 46 feet. The piles were driven by a heavy steam-hammer with the aid of a 40-foot follower to such an elevation that they could be cut off at 36 feet below low water. The piles were located on intersecting lines with unusual accuracy, the pile-driver being fitted with sliding extension leads 88 feet long.

Forms of pile records differ more or less according to the character of the structure which they are to support. The American Railway Engineering Association adopted a standard pile record form for railroad trestles, after the examination by a committee of a large number of forms used by different railroads. It is published in the Manual of the Association. The four lines above the tabular form are for the bridge number or name, its location, the weight and kind of hammer, the date, and a statement that bents are numbered from the north or east end, and that piles are numbered from left to right. The column headings are as follows: Date; Bent Number; Number of Pile; Size of Pile, Including Tip End, Butt End and Length; Kind of Wood; Length of Cut-off; Distance from Base of Rail to the Ground; Total Penetration; Average for the Last Five Blows, of the Drop of the Hammer and of the Penetration; Kind of Soil; and Remarks.

In foundations of buildings, bridge piers, wharves, etc., the piles are usually arranged so that the rows in one direction can be lettered *A*, *B*, *C*, etc., while the piles in each row are numbered. Any pile can thus be designated by a letter combined

with a number, thus: $E19$. In the foundations of pivot piers and of circular buildings the piles are preferably arranged in circular rows, which may be similarly designated by letters.

ART. 40. SPECIFICATIONS

When the extent to which timber pile foundations have been employed in engineering practice in America is taken into account, it is remarkable that the specifications provided for this part of the construction, with but few exceptions, have been so inadequate. As a rule, they referred briefly to the minimum size, and a few other qualities of the piles that would be accepted, and added a clause regarding the required penetration of the piles, which in most cases could not be carried out without damage to the piles, because the subsurface conditions had not been explored. They usually omitted any reference to the column action of the piles, and gave no permissible unit-stresses.

There is one expression which should not be used in modern specifications, and which has probably been responsible in the past for more caustic criticisms on the part of contractors and more unjust flings at inexperienced inspectors than any other paragraph in the whole range of engineering specifications. The expression is "practical refusal," which was doubtless expected by the authors of specifications to define the limit to which piles were to be driven. The term, used generally without explanation, has been interpreted more or less literally by youthful inspectors. In reality it is an ambiguous term, since one engineer writes "practical refusal of, say, 1 inch;" another, "practical refusal—that is, until the pile does not penetrate more than ½ inch under a 2000-pound hammer falling 15 feet"; and still another, "to a refusal of a 2000-pound hammer falling 20 feet or its equivalent"; while a fourth modifies it slightly thus: "to a good refusal under a hammer weighing 1500 pounds falling freely 12 to 15 feet, or its equivalent." It will be noted that only one of these statements is definite. If an inexperienced assistant is to be assigned as inspector it would accord with good practice in other departments for the engineer

in charge to provide him with suitable instructions so that the contractor may be treated with fairness.

The larger parts of the specifications for timber piles adopted by the American Railway Engineering Association were quoted in Arts. 3 and 4. The following specifications on piles and pile driving are reprinted, by permission, as being one of the most rational and the most complete specifications on the subject. The paragraphs relating to concrete piles are given in Art. 57.

Extracts from General Specifications for Bridges. Part III, Substructures and Concrete Bridges, by J. E. GREINER, 1911.

86. Piles shall not be used for foundations unless a penetration of at least 12 feet in firm ground or 30 feet in soft ground is assured by the character of the underlying strata. They shall generally be spaced not closer than 2 feet 6 inches center to center and their tops should be imbedded at least 6 inches into the concrete footing course of the masonry. When they pass through water or soft ground before entering firm ground, their strength as columns shall be considered as well as their supporting power due to friction. When subjected to transverse forces, batter piles shall be driven in sufficient numbers to resist the transverse forces without assistance from the vertical piles. For foundations of arch or movable bridges or high abutments the piles shall be completely embedded in firm earth, sand or gravel which will afford good lateral support. When this is impracticable then the soft material shall be excavated to a depth of at least 12 feet and heavy stone riprap used for stiffening and protection.

87. Timber piles, unless treated, shall not be used as an essential part of the foundations above the ground or in ground not permanently wet or in water infested with wood borers. When of the quality and dimensions hereinafter specified and when driven to practical refusal, the direct load on any timber pile shall not exceed 16 tons for railway bridges, all movable spans, arches and high abutments or 20 tons for other foundations. When the piles are in water and act as columns, the maximum stress per square inch at the center of the unsupported length shall not exceed that permitted by the following formula:

$$S = S'\left(1 - \frac{l}{60d}\right).$$

Where S = permissible stress per square inch at center of unsupported length; S' = 600 pounds for longleaf yellow pine or white oak, and 500 pounds for Douglas fir or northern pine; l = unsupported length in inches; d = diameter at center of the unsupported length in inches. The above

is applicable to lengths between 15 and 30 times the diameter at center of unsupported length. When piles are not in water but are exposed to view, the above working stress for columns may be increased 50 percent.

132. Where piles are used in the foundations the soft silt or mud shall be excavated to a stratum of sufficient firmness to give lateral support and for important structures such as piers for movable bridges, arches and high abutments the space around and between the piles shall be filled with riprap or gravel as indicated on the plans or as may be required by the engineer. When not indicated or specified, the cost of riprap shall be paid by the company. Piles shall not be used for any important work, even when called for on the plans, unless there can be obtained a penetration of at least 12 feet in firm material or 30 feet in soft ground, or unless their use is authorized by the engineer after the conditions are fully known, in which case he shall determine the number required in addition to those shown on the plans and the location of the brace or batter piles necessary for lateral support. Piles shall be located accurately in the positions indicated and cut off at the required elevation. They may be driven either by gravity or steam-hammers, but shall have their butts protected with metal bands, cushions or other means for preventing damage, and shall be handled and driven in a manner that will insure them against injury. Where the strata are of such a nature that driving is liable to injure the piles, they may be jetted down to solid ground. Before any piles which are to remain in the completed structure are ordered or driven, the contractor shall determine the length required by driving a sufficient number of test piles for this purpose. In case he fails to do this, piles ordered by him of insufficient length for proper driving shall be at his risk.

133. Timber piles shall be of sound longleaf yellow pine, Douglas fir, cypress or white oak, butt cut above the ground swell, from sound trees when the sap is down, close-grained and solid, free from injurious ring shakes, large, unsound or loose knots, or other defects which may materially affect their strength or durability. They shall have a uniform taper from butt to tip, be free from short bends, and a line drawn from the center of the butt to the center of the tip shall lie wholly within the body of the pile. They shall be peeled soon after cutting, and all knots trimmed close to the body of the pile. Unless otherwise indicated on the plans, the minimum diameter at the tip shall be 9 inches for lengths up to 30 feet, 8 inches for lengths over 30 feet but not exceeding 50 feet, and 7 inches for lengths over 50 feet. The minimum diameter at one-quarter length from the butt shall be 12 inches and the maximum diameter at the butt 20 inches. When treated piles are required in water infested with wood borers the treatment shall be with dead oil of coal tar or other acceptable process to the extent of 20 pounds per cubic foot unless otherwise specified.

134. When driven through hard ground they shall be shod with steel points of approved design. They shall go to rock or to practical refusal

which is here understood to mean driven to such depth that the last five blows of a 3000-pound hammer freely falling 15 feet upon the solid unbroomed head of a pile shall not produce an average penetration greater than ½ inch for each blow. For other weights of drop-hammers falling from 12 to 15 feet and for steam-hammers, the penetration for practical refusal as above defined may be determined from the following formulas:

(*a*) Gravity hammers:

$$s = \frac{WH}{30{,}000} - 1.0; \text{ average for each of last five blows}$$

(*b*) Steam-hammers:

$$s = \frac{WH}{30{,}000} - 0.1; \text{ average for each of last 20 blows}$$

where s = penetration in inches; W = weight of the falling hammer in pounds; H = height of the fall in feet. In case the above refusal cannot be obtained without injury to the pile or on account of the impracticable lengths required, the number indicated in the plans shall be increased until the maximum load coming on any pile shall not exceed that determined from the formulas:

(*a*) For railway bridges, all arches and movable spans

$$P = \frac{1.06WH}{s+1} \text{ for gravity hammer; } P = \frac{1.06WH}{s+0.1} \text{ for steam-hammer}$$

(*b*) For other structures the above loads may be increased 25 percent.

CHAPTER IV

CONCRETE PILES

ART. 41. INTRODUCTION AND CLASSIFICATION

Being impressed by the very short life of timber piles when their upper portions are alternately wet and dry, as in pile trestle bridges, the increasing cost of timber, and the decreasing cost of cement, A. A. RAYMOND was led to consider the design and construction of concrete piles. He first used such piles in 1901 in a building foundation in Chicago.

In 1897, HENNEBIQUE introduced the reinforced-concrete pile in Europe, and in 1904 it was first used in America. Within the first decade of this century, a number of other forms were developed, differing in methods of construction, some of them being patented by the inventors, while others were designed by engineers, without the use of patented material, form or arrangement.

Concrete piles may be divided into two general classes: The first class comprises those which are molded to a regular form, and, after curing, are handled and driven like timber piles; while the second class includes those formed in place either with or without the use of casings which remain until destroyed by corrosion. The former may be called pre-molded piles and the latter, cast-in-place piles. Pre-molded piles were first developed in Europe and practice in that country has practically confined the use of concrete piles to that class. The cast-in-place piles were invented in America and on that account came into considerable use before the advantageous features of pre-molded piles came to be generally recognized.

Pre-molded piles are always reinforced with steel bars or rods in combination with lateral reinforcement in the form of wire hoops or spiral wrapping. They are square, hexagonal,

octagonal or circular in cross-section, the corners of square piles being chamfered, however. Piles with a circular cross-section generally have no taper, while the others are usually tapered from butt to tip.

This class includes several distinct types. The Chenoweth pile is formed by rolling it in a machine especially designed for the purpose, the reinforcement being arranged, so as to show a spiral form in cross-section. The corrugated pile is octagonal in section but with a semicircular corrugation on each face intended to increase the surface for frictional resistance, and with a hole along its axis.

The Cummings pile is distinguished mainly by the character and arrangement of the reinforcement, which is electrically welded and handled as a unit. In some cases, annular grooves are molded on the surface. The preceding types are patented, but most of the pre-molded piles are unpatented and simple in construction, as illustrated in Art. 43.

Cast-in-place piles are usually not reinforced, although this feature may be added. The type in most extensive use at present (1925) is the Raymond pile. It is made by first driving a steel collapsible core with a sheet-iron form, which remains in place after the core is withdrawn, and is then filled with concrete. The simplex pile is made by driving a steel pipe with movable point into the earth and filling the hole with concrete, as the pipe is gradually withdrawn. The pedestal pile differs from the simplex in having an enlarged foot formed by means of a plunger which forces the concrete to make and fill an enlargement in the hole during the withdrawal of the pipe for several feet from the bottom. In the peerless sectional pile, sectional concrete casings are driven into place and afterward filled with concrete. Practically all cast-in-place piles are patented. Their construction is described and illustrated in Art. 47.

In the bulkhead construction for the new port at San Diego, Cal., a combination concrete and timber pile was used. A wooden pile 14 inches in diameter was driven into the harbor bottom, a hollow concrete cylinder cast on shore was then driven

over the pile to a depth of 4 or 5 feet below the bottom of the harbor, and filled with concrete.[1]

ART. 42. RELATIVE ADVANTAGES

Timber piles in ordinary foundations must be cut off below the permanent ground-water level, which often involves the cost of extra excavation. When the water-level is lowered by changes in the drainage system due to the construction of subways, the lowering of sewers, or for other causes, the piles become liable to rot and may involve expensive changes in the foundation.

The durability of concrete piles is independent of the ground-water level. Concrete piles have a material advantage on account of their greater bearing capacity, due to their larger size, thus permitting a material reduction in the number required to support a given structure. The bearing capacity may be further enlarged under some conditions by increasing the taper. Roughly, the loading of timber piles is restricted by engineers within a range of 10 to 20 tons, while concrete piles may be loaded from 20 to 50 tons.

On the other hand, a concrete pile costs considerably more than a timber pile. Since the life of concrete piles is not dependent upon the ground-water level, their use not only avoids extra excavation, but as a direct consequence a saving in the masonry walls or footings. Most frequently, this is the largest factor in saving afforded by the substitution of concrete piles, provided the tops of the concrete piles can be placed more than 3 feet higher than for timber piles. They may also be readily bonded into the grillage or capping of concrete, and will act together as a monolith, provided reinforcement is used in the piles. Less excavation and smaller footings imply a reduction in the time of construction.

When pre-molded piles are employed, they require more time and care in handling than timber piles, on account of

[1] For additional details and illustrations see Engineering News, vol. 69, page 498, Mar. 13, 1913.

their greater weight, and their relatively lower flexural strength. In general, concrete piles cannot be driven as rapidly as timber piles, but the number required may be sufficiently smaller to effect a saving in time, as well as in cost. As sand, stone and cement are generally available, there is less probability of delay, occasioned by waiting for the arrival of piles at the site. Sometimes the value of the time saved pays for a considerable part of the piles. In some track-elevation work in cities, the use of concrete piles for the foundations of retaining walls has made a large saving by reducing the required width of new right-of-way at the excessive rates which had to be paid.

In emergencies, concrete piles have sometimes shown unusual flexural strength. In one case, a single pile acting as a cantilever, 32 feet long, successfully withstood the test, when a 9-inch hawser was attached to its upper end to pull a steamer of 4800-tons displacement to the unfinished pier against a rapidly running tide. At another time, a steamer ran into the pier by accident, and broke off a number of pine piles, but none of the concrete piles. It may be added, that concrete piles may be placed in some filled material through which it is impossible to drive timber piles without injury. Strict economy requires that adequate exploration of the soil be made to determine the proper lengths of piles. Failure to do so, leads to waste of timber by excessive cut-offs, but with concrete piles the waste of time may be even more serious than that of material and labor.

Inspectors realize how difficult it is often to find a fair percentage of timber piles which fill the demands of the specifications in all particulars, regarding diameter of butt, diameter of tip, straightness or other qualities, especially when the required length exceeds 50 feet. On the other hand, with reasonable care, every concrete pile can be made to comply fully with the specifications. Moreover, the strength of concrete piles improves with age.

As the forest resources are being reduced, it becomes increasingly difficult to get the larger sizes of timber piles, while at the same time the quality of the wood becomes poorer. Although

the safe allowable compression for concrete is less than for wood on the ends of the fibers, the loading of a pile depends more frequently on the supporting capacity of the earth than on the strength of the pile.

There are minor advantages that can only be adequately considered in the detailed design and estimate of cost for a given structure. Any decrease in the masonry of walls or footings has a secondary influence on the cost of the foundation by reducing the weight to be supported, provided this is not wholly offset by the increased weight of concrete piles. Similarly, a reduction in excavation may lessen the amount of sheeting, shoring, pumping, back-filling, etc. These items are often difficult to estimate and hence contractors, as well as engineers, prefer to eliminate them, if possible.

In the following example, the cost with concrete piles is greater than for concrete piers in shafts sunk to rock, but the piles had to be adopted since a stratum of fine water-bearing sand was discovered when an unsuccessful attempt was made to put down the first shaft of bent 15. The statement below gives a comparative estimate of cost of foundations for tower No. 15-16 of the Municipal bridge approach at St. Louis, built in 1911.

	Concrete piers		Concrete piles	
Concrete, cubic yards ...	302 3 @ $5 70	$1725	127 8 @ $5 70	$728
Excavation, cubic yards	325 5 @ 1.75	570	216 4 @ 0 60	130
Concrete piles, linear feet	....	....	1550 @ 1 10	1705
Total for six piers ...	....	$2295	....	$2563

As indicated on the boring sheet, the average depth of rock below the ground surface was about 31 feet. The yardage cost of concrete includes cost of forms; and that of excavation includes sheeting, shoring and pumping. The reinforced concrete piles were driven through strata of filled ground, shore sand, blue sandy clay, coarse sand and gravel, and river sand to rock.

In salt water infested by the Teredo or Limnoria, the timber pile requires expensive protection either by chemical treatment (Art. 23), or by mechanical means (Art. 24). The concrete pile is wholly free from the ravages of these borers.

The following example illustrates the method adopted to determine the comparative cost of foundations designed for the use of timber and concrete piles respectively. On account of increase in traffic and live loads, causing settlement of the footings supported by piles, and requiring an additional track later, substantial reconstruction became necessary for the viaduct foundations of the Norfolk and Western Railway approach viaduct at Kenova, W. Va. Two designs were made to determine which was the more economical to construct. In one plan three rows of four creosoted piles each with four caps were placed under concrete footings, while in the other plan concrete piles were used under each footing, three piles being placed in each of the two outer rows and two in the middle row. The comparative estimates of the average cost of each footing were as follows:

DESIGN WITH TIMBER PILES

31 cubic yards excavation @ 50 cents	$ 15 50
22 cubic yards portland cement concrete @ $6 50	143 00
348 feet B M creosoted timber @ $45	17 28
240 feet creosoted piles @ 56 cents	134 40
55 pounds iron @ 6 cents	3 30
	$313 48

DESIGN WITH CONCRETE PILES

20 cubic yards excavation @ 50 cents	$ 10 00
14 cubic yards portland cement concrete @ $6 50	91 00
8 concrete piles @ $22	176 00
	$277 00
Estimated saving on each footing	$36 48

The concrete piles were 20 feet long, 20 inches in diameter at the head and 6 inches at the foot. On account of the size and taper of the concrete piles a smaller number could be used. The reduced amount of excavation and of concrete in the footings effected the balance of the saving in cost. The size of the con-

crete cap and footing was in both cases 4 by 6 feet on top and 8 by 10 feet on the bottom, but the height was reduced from 11 to 6 feet.

Art. 43. Unpatented Pre-Molded Piles

A pre-molded pile is a reinforced concrete pile which is molded to a regular form and after curing and seasoning is handled and driven like a timber pile. In order to indicate the principal variations in form and reinforcement which have been developed by different designers of pre-molded concrete piles, brief descriptions are given either of standard designs or of those adopted for the foundations of particular structures.

The bridge department of the Chicago, Burlington and Quincy Railroad was a pioneer in the design and construction of low reinforced-concrete pile trestles for steam railroads. In connection with the thorough studies and tests made for this purpose, an unpatented type of pre-molded pile was developed in 1905, which, together with the Chenoweth rolled pile (Fig. 43*b*), has been extensively used in construction by that railroad. The piles used in the first of these bridges are 16 inches square at the butt, have a 4-inch chamfer at each corner, a taper of 4 inches in 30 feet on each face, and are pointed at the tip. The reinforcement consists of four ¾-inch square corrugated bars, hooped with No. 12 steel wire, wound at close pitch near the butt and tip, and at 3-inch pitch over the greatest part of the length of the pile.

Since the cost of making the reinforcement units was one of the principal items, experiments were made to reduce it by molding a pile without taper, and using a wire netting which could then be simply folded into a square prismatic form and wired together at the lap, thus greatly lessening the labor involved. The cost of forms was thereby also materially reduced. Figure 43*a* shows the details of the form and reinforcement of this later design. It will be observed that the corners of the pile are rounded and that the longitudinal bars are wired to the fabric at its corners. At the point, the transverse wires were cut and

the longitudinal wires brought together and tied securely with small wire.

Reinforced-concrete piles were designed by the Pennsylvania Lines for their extensive docks at Cleveland, Ohio, and were constructed by the Great Lakes Dredge and Dock Co. They are octagonal in shape, without taper, pointed at the foot and have a cast-iron shoe which was made an integral part of the pile. As indicated in Fig. 43*d*, the reinforcement consists of eight longitudinal rods securely bound together at regular intervals throughout the body of the pile by tie rods. They are also spirally wrapped for short distances at both head and foot. The dimensions in Fig. 43*d* refer to piles 30 to 40 feet long, the longitudinal rods being 1 inch in diameter, while the ties and wrapping are ⅜ inch in diameter. Over 3500 octagonal piles were used on the dock foundations and all of them were cast in vertical molds. The weight of the largest pile is 6 tons.

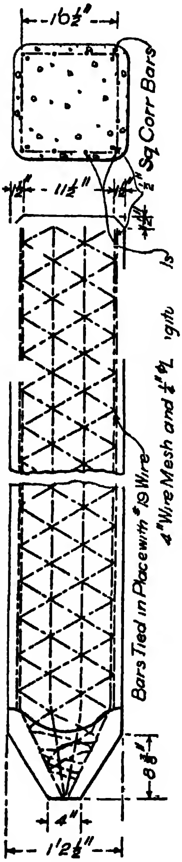


FIG. 43*a*.

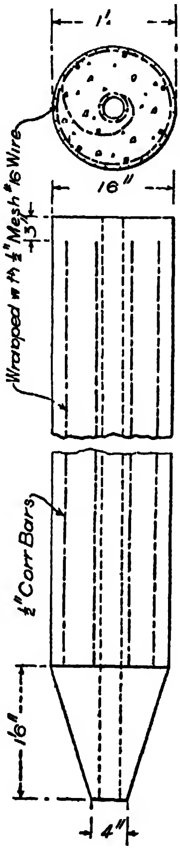
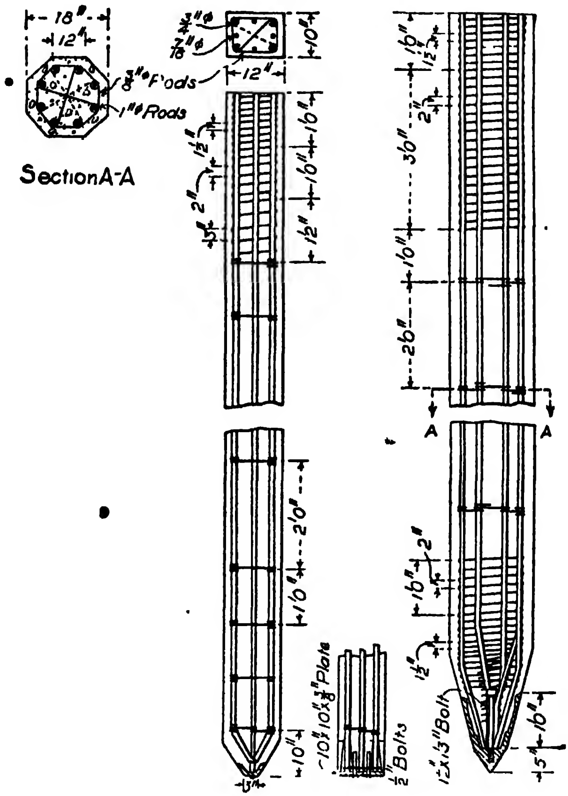


FIG. 43*b*.

The standard design of the Chicago, Rock Island and Pacific Railway, adopted in 1910, is illustrated in Fig. 43*e*. The form is octagonal, without taper and pointed at the foot. The reinforcement occupies a cylindrical form in the body of the pile, and consists of six longitudinal corrugated bars and spiral wrap-

pings of wire with a small pitch, which is slightly modified near the head and foot. The bars extend to the point but not quite to the upper end, and are wired to the helical reinforcement at intervals not exceeding 12 inches.

A similar design was adopted for some pile foundations in the approaches of the Municipal bridge at St. Louis, Mo., in 1911.



FIGS. 43*c* and *d*.

The least diameter is also 14 inches and the reinforcement consists of six ⅝-inch round rods, and a helical winding with No. 9 wire on a pitch of 4 inches, reduced to 1 inch for about 18 inches at the head of the pile.

The pre-molded piles designed for the Government pier at Halifax, N. S., are 24 inches square in section with the corners

slightly chamfered. The eight reinforcing rods are 1 inch in diameter for piles under 60 feet in length, 1⅛ inches for lengths of 60 to 70 feet, and 1¼ inches for lengths over 70 feet. In addition to these rods, which extend from the pyramidal point to 3 feet above the head so as to bond into the superstructure, four rods 8 feet long extend from the point upward about midway between the axis and the full-length corner rods. Helical

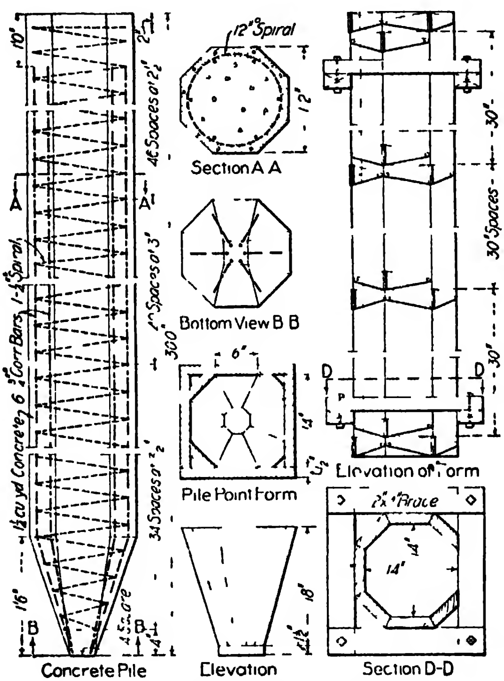


FIG. 43*e*.—Concrete Pile and Forms.

wrapping with ¼-inch wire and a pitch of 2 inches is used for a distance of 5½ feet above the point. Beyond that the wire hoops are spaced 12 inches apart except at the head, where the spaces are reduced to 9 and 6 inches respectively.

The reinforced-concrete piles used in 1906–07 in the foundations of the steamship terminal of the Atlanta, Birmingham and Atlantic Railway at Brunswick, Ga., were tapered for a length of only 10 feet from the tip. Their length ranges from 30 to 51

feet and their weight from 3 to 5 tons each. They are 16 inches square from the shoulders to a section 10 feet from the tip and 8 inches at the tip. Above the shoulders, they have a projecting tenon, 8 by 16 inches and 16 inches long to permit concrete fillers to be placed in the shoulders to increase the bearing area of the packed timber beams used in the floor of the pier. The reinforcement consists of four 1¼-inch bars placed in the corners of the pile and tied together at 12-inch intervals by ¼-inch round steel clips. At the head electric-welded wire fabric is placed for a length of 4 feet to aid in withstanding the shock from

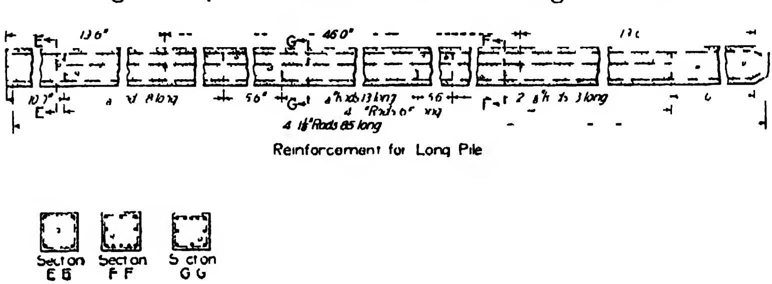


Fig. 43f Design of 85-foot Pile

hammering. A 1½-inch jet pipe was cast into the lower part of the pile for subsequent use in sinking.

Figure 43f shows the design of an 85-foot pile used in dock construction in Havana. The section is 20 inches square, the reinforcement being proportioned for the bending moment when the pile is supported at two points located to make the maximum positive and negative moments equal, approximately the one-fifth points. Transverse gas pipes were inserted at these points to provide a convenient method of handling the piles. The piles were hooped with ¼-inch square rods and had a special basket reinforcement at the top to take the hammer impact.

Concrete piles 100 feet long were used in the construction of wharves in New Zealand. These piles were 20 inches square and reinforced with eight 1½-inch rods. In handling the piles they were supported at four points, at each end and near the third points.

To reduce the amount of reinforcement the five-point suspension is sometimes used on extremely long piles. A good example of this is given in the England News-Record, vol. 80, page 659, Apr. 4, 1918. The piles were 100 feet long, 20 inches square, with 1⅛ -inch square bars at each corner and additional reinforcing near the middle of the pile.

Figure 43g shows a 24-inch square pile being driven for the Woodhaven Boulevard extension in New York. The maximum length of piles on this job was 105 feet and the maximum weight 30 tons. The reinforcement consisted of 1⅛-inch square bars and ½-inch round bars, the volume of steel being 1.87 percent that of the concrete.

Pre-molded round piles, 12 inches in diameter but having a bulb-shaped foot 30 inches in diameter and 24 inches high, were used on some new work for the reconstruction of the old steel pier at Atlantic City into a reinforced-concrete pier. The longitudinal reinforcement consists of six ¾-inch bars, and these are splayed out at the bottom to reinforce the foot, which is intended to increase the bearing power in the sand. A 2-inch jet pipe was cast in the pile and extends throughout its full length.

On pages 418 and 419 of the Manual (1921) of the American Railway Engineering Association will be found four typical designs of pre-molded piles.

ART. 44. PATENTED PRE-MOLDED PILES

The corrugated pile is hexagonal or octagonal in cross-section with grooves approximately semi-cylindrical on each face, has a round hole along the axis, both pile and hole being tapered from butt to tip, and is reinforced with electrically welded wire fabric (Fig. 44*a*). The hole or central bore is used to permit a water-jet to pass through it, and it is tapered to increase the section area of concrete at the tip and to permit the plug used to cast the hole to be easily withdrawn. The object of the corrugations is to increase the pile surface for skin friction, and to furnish convenient outlets for the escaping water from the jet,

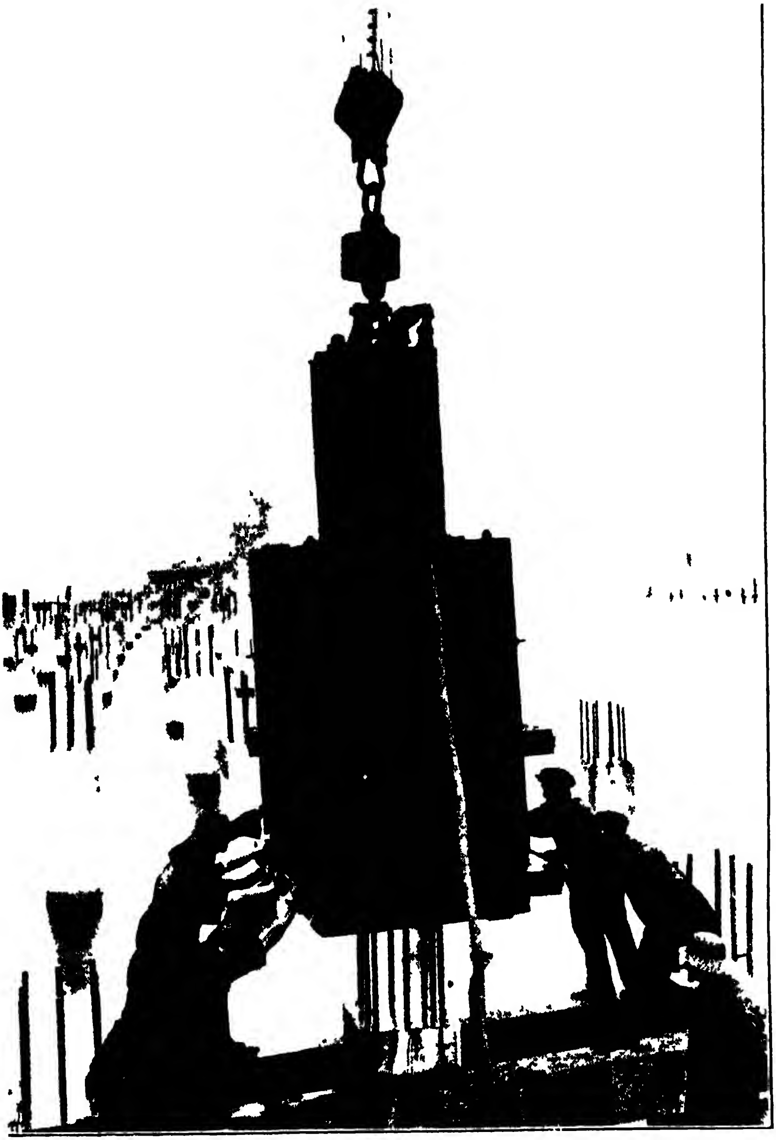


FIG 438 —Driving 30-ton Piles.

(*Facing p. 132.*)

thus reducing the friction during the operation of sinking. They are not extended, however, along the head or tip, to avoid reducing the full section area of both parts. When the piles are intended to project above the ground level, the corrugations are designed not to extend above it.

The Chenoweth concrete pile is a rolled pile. The machine used for this purpose has a moving platform, a number of rolls and mechanism to turn the tubular mandrel, about which the pile is formed. The reinforcement consists of a number of longitudinal corrugated bars, wired to transverse strips of wire mesh. This reinforcement is laid out on the platform and the

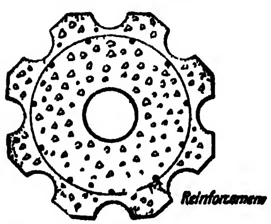


FIG 44a — Section of Corrugated Pile

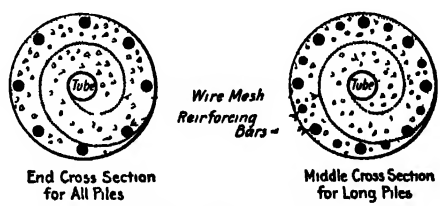


FIG 44b — Sections of Chenoweth Pile.

ends of the wire mesh attached by wire clips to the keyways of the mandrel; the concrete is properly spread over it and then by simultaneously turning the mandrel and moving the platform, the pile is coiled and rolled into a cylindrical form which is compacted and shaped by means of the adjustable rollers. At the same time, the pile is wound by wire at about 6-inch intervals during the entire process of rolling. After fastening the ends of these wires, the central tube is withdrawn, and the pile is removed to the drying table. A concrete point shaped like the frustum of a cone is constructed around the projecting reinforcing bars with the aid of a suitable form, and the head is perfected in a similar manner. The wire netting in the finished pile is located in a spiral surface, as indicated in the cross-section shown in Fig. 44*b*, while the longitudinal bars are equidistant near the surface of the pile. In making these piles, a very dry mixture has to be used; otherwise, the cement will be squeezed out with the water in rolling. If the mixture is too wet, the

piles will also lose their shape and become oval on the drying table. On some railroads, it is the practice to omit pointing the rolled piles, leaving the tip, just as it comes from the rolls.

The largest pile of this type employed prior to 1913 was used in a coal dock at Havana, the length being 76 feet, diameter 18 inches and weight 12 tons At the end of the dock, the piles are unsupported for a length of 40 feet and their penetration extends 2 feet into the coral rock. Piles 50 feet long and only 12 inches in diameter have been successfully handled Piles of the same length but 14 inches in diameter and weighing $3\frac{1}{2}$ tons have been hauled 4 miles to the site of the foundation

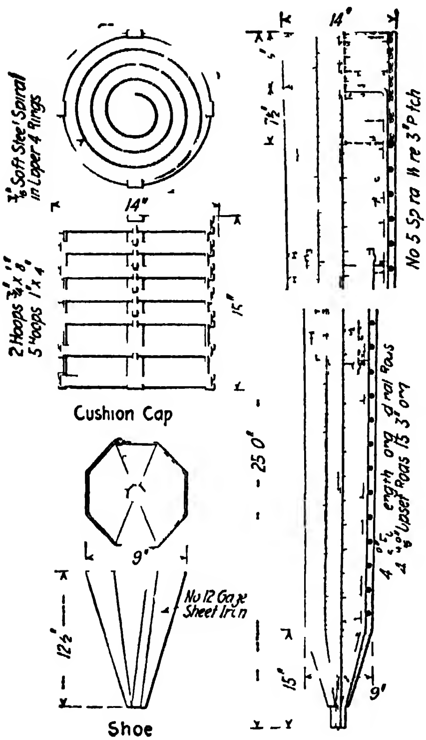


FIG. 44c —Reinforcement of Cummings Concrete Pile

The reinforcement of the Cummings concrete pile is illustrated in Fig 44c Four of the longitudinal bars do not extend the full length of the pile. The other four are welded together at the point and welded to a conical sheet-metal point protector or shoe. They are also bent over at the head and welded together in pairs. The longitudinal rods are held in position, at intervals of approximately 5 feet by flat rings of $\frac{1}{4}$-inch metal with notches cut in the circumference to receive the rods. The body of the pile has a helical wrapping or wire to perform the function of hooping and to aid in resisting diagonal stresses

At the head there is also a series of horizontal bands closely spaced to give special lateral support to the concrete under the

driving shock and each of the upper four of these contains a horizontal spiral as shown in the sketch plan. Sometimes seven are used in large piles. This arrangement for reinforcing the pile head has proved to be very resistant to the impact of the pile-driver hammer. It is claimed, that, in driving thousands of piles with this resilient head, in no case was the head of the pile broken.

Figure 44*c* shows the tapered form which is always made with an octagonal cross-section and a tip 9 inches in diameter, the diameter of the butt depending upon the length of the pile so as to preserve a constant taper of 2 inches in 10 feet in the standard designs. When a pile of uniform section is used, the form of the cross-section is circular. Sometimes both the tapered and untapered piles are molded with annular grooves to increase the frictional resistance.

The ordinary form of the Hennebique pile is usually constructed as a square pile without taper, with the reinforcing bars near the four edges, which are slightly beveled and tied together by wire collars or binders at short intervals. The standard form has a cast-steel shoe at the foot forming an integral part of the pile. To support a reinforced-concrete quay, at Key West, piles 16 and 20 inches square and from 25 to 60 feet long were driven through marl and sand into the coral rock, all of them being provided with metal shoes. The longitudinal reinforcement consisted of four rods 1⅛ inches in diameter, with extra rods in the middle third of the longest piles. The ¼-inch wire collars were spaced 12 inches apart in the body of the pile, 4 inches in the head, 6 and 3 inches in the tip. A pile of this kind, which was constructed as a hollow pile to reduce its weight, and was filled with concrete when in place, is described in Art. 49.

The Jones-Bignell pile is designed to be placed without driving, sinking being effected entirely by the water-jet. This pile has a 4-inch pipe through its entire length, this diameter being reduced by means of a special nozzle to 1⅛ inches at the bottom. This 4-inch pipe is tapped at intervals throughout its length with pipes of from ¾- to ⅜-inch diameter, the latter

being fitted with elbows turned up on the sides of the piling. In addition, a 2-inch pipe extends through the 4-inch pipe and fits into the 1⅛-inch nozzle at the bottom. The 2-inch pipe carries a water pressure of 200 to 300 pounds per square inch, while the larger pipe carries a pressure of about two-thirds this. The jet at the bottom of the pile loosens and displaces the material under the pile point, while the jets on the sides keep the soil loose and overcome side friction.

Further details as to sizes of piles, sizes and disposition of reinforcement and methods of construction may be found in the elaborately illustrated catalogues which are published by the construction companies.

ART. 45. FORM AND CONSTRUCTION

The prevailing form of cross-section for pre-molded piles is octagonal. Practically all of those with square sections approximate toward the octagonal form in that their edges are beveled to a width of 2 or 3 inches, although occasionally they are merely rounded (see Fig. 43*a*). The circular cross-section is used but seldom, in which case the piles must be cast in a vertical position. The diameter varies from 10 to 25 inches, but it is rarely below 12 or above 18 inches. The length varies from 8 to over 100 feet, but it is questionable whether any length less than 15 feet should be employed in any pile foundation. In most cases the length ranges from 20 to 40 feet. The longest piles are used in dock construction where the piles are located in deep water, the longest on record (1925) being 110 feet (Art. 50).

It would be difficult to say whether more pre-molded piles are constructed with taper than without it, for both forms are in extensive use. A number of railroads have adopted the straight or untapered pile as standard, and in a considerable number of important works, each one requiring thousands of piles, half of them use the form with uniform cross-section. This form should always be used when conditions require the pile to act chiefly as a column.

The tendency is to use a decidedly smaller taper in pre-molded piles than that for cast-in-place piles. It may be stated that

except for very short piles in the foundations of buildings, the taper does not exceed 1 inch in 4 feet, is frequently 1 inch in 5 feet and sometimes as low as 1 inch in 7.5 feet. The influence of taper on the bearing power of piles is discussed in Art. 55.

As indicated in Arts. 43 and 44, pre-molded piles are most frequently designed with a point at the foot. Its length often exceeds but is sometimes less than the diameter. Even in tapered piles experience has shown the advantage of a point. In driving piles for the Kentucky shore pier of the Kentucky and Indiana bridge at Louisville in 1910, the 9-inch tip of the pile was made square-ended to insure straight driving. But as driving through the hard clay proved to be so difficult, the last batch of piles for the pier footing was provided with pyramid points, which increased three fold the number driven per day.

In some experiments reported in 1908 made by THOMPSON and FOX with tapered piles 30½ feet long it was found that the 8-inch tip required less time to drive the piles than 9- or 10-inch tips, although in this case the result was complicated by the effect of difference in taper. The use of metal shoes in pre-molded piles is referred to in Arts. 43 and 44 (see Fig. 43*d* for an illustration of one form).

Sometimes the foot of the pile has been enlarged in diameter in order to increase the bearing area of the pile in the lower stratum. One example of this practice is found in the 16-inch reinforced concrete piles for the Atlantic City boardwalk, built in 1908, which were formed with a base 26 inches in diameter (see also Art. 43).

It is customary in good practice to fabricate the reinforcement as a unit, so that it can be easily handled and placed quickly in the form when the process of casting is under way. The reinforcement unit is held in accurate position in the forms by suitable hangers and separators, so that the conditions assumed in designing the pile shall be realized in its construction.

If the pile is to have a hole in the center for the insertion of a jet pipe to be used in sinking, which is more economical than to cast a jet pipe in the pile, either a tapered wooden core may

be used, or preferably a collapsible form; or a tin tube may be used instead and left in the pile. The objection to the solid core is that it requires occasional turning to prevent its sticking to the concrete, and its removal later.

The composition of the concrete consists of 1 part portland cement, 2 parts of sand and 4 parts of broken stone or gravel. This mixture is so generally employed that it may be regarded as standard. Occasionally it is modified to 1-2-3. In large hollow piles the molded portion may have a composition of 1-1.5-3 and a leaner mixture like 1-3-5 or 1-3-6 employed in filling the interior after they are in place. Usually the size of the crushed stone or gravel is limited to ¾ inch. Investigations relating to the effect of sea water on concrete piles have not resulted in definite conclusions. Experience has shown, however, that it is important to make as dense a mixture of concrete as possible, and mixtures as rich as 1-1-2 have been used. In the work illustrated in Fig. 43g the concrete was protected by jacketing from an elevation 2 feet below low water to 3 feet above high water with a solid casing of two thicknesses of 3-inch creosoted planks, secured by galvanized ties 3 feet apart vertically.

At first piles were molded in forms laid horizontally on the ground or on suitable platforms. Later the practice of molding piles in a vertical position was introduced in order that the surface of the concrete as it is deposited in batches shall always be perpendicular to the direction of the load to be supported by the pile, or to the force applied by the hammer in driving. When piles are molded in a horizontal position, special care should be exercised to provide an unyielding base so that the concrete may not be subjected to flexural stresses while in the process of setting. When the forms are vertical, special precautions must be observed in tamping or puddling the concrete to eliminate all voids. In one case where 361 piles were cast vertically in lengths of 28 and 32 feet, not one unsound pile was found when the forms were removed. Both positions of the molds have been used on large constructions where adequate equipment was provided.

With horizontal molds the sides may be removed in from 24 to 48 hours, but the pile is allowed to rest on the base about a week longer, during which time it should be copiously showered with water to permit complete chemical action for the setting of the cement. In very warm weather some protection from the sun may be required. After this the piles may be removed and piled in stacks to continue seasoning, using an equalizing spreader and bridle, if necessary to handle them. They are usually allowed to harden for at least three weeks more before they are driven. The actual time to be allowed in each case depends upon the temperature and humidity of the atmosphere, while the age at which piles may be placed in position depends also upon the character of the ground and the method of driving.

The hardening of the concrete may be materially hastened by curing with live steam under cover. The piles for the docks of the Pennsylvania Lines at Cleveland were cast vertically in steel forms. The pile forms were then closed at the top and placed horizontally on a floor of cross timbers. On account of the late season, it was found necessary to use some method of artificial seasoning and this was done by piling up the newly filled forms and covering them with canvas. A steam-pipe line provided with outlet pipes was laid to discharge steam under the cover, and to maintain a temperature of about 80 degrees. The forms were removed in from 12 to 18 hours, thus leaving the concrete piles exposed directly to the steam for three or more days afterward until they were set sufficiently to be handled by a derrick. They were afterward placed by derricks in the storage yard to be kept at least 30 days before driving.

It may be added that on other works concrete piles have been allowed to set for from five to six days in the ordinary manner and then gently hoisted to the curing bed with 25 or 30 stacked together in a pile with wooden spacing blocks between them, when they were subjected to live steam for two or three days. They were then driven within three or four days in summer, or within ten days in winter.

As soon as pre-molded piles are driven, they are ready to receive their load from the superstructure above.

ART. 46. DESIGN OF PRE-MOLDED PILES

The steel reinforcement of a concrete pile is intended to resist the stresses due to handling and driving the pile, and to the load which may come upon it in its final position. The longitudinal bars receive their greatest stresses when the pile is lifted from a horizontal position. Unless the pile is exceptionally long or heavy it is often picked up at or near the middle in going to or from the seasoning yard, or a line may be attached near one end to drag it to the pile-driver. In the former case the pile must be strong enough to resist flexure due to its own weight, while in the latter case the pile must not only sustain its own weight but also shock or the impact due to meeting obstacles, which sets it into active vibration.

When so handled, concrete piles rarely fail by compression, but cracks develop on the tension side which sometimes may be due to the slipping of the rods. On plain versus deformed bars, see Bulletin 71, Engineering Experiment Station, University of Illinois. Large cracks may endanger the permanency of the reinforcement by permitting corrosion to occur. It is also important that uniform circumferential spacing of the bars be maintained in construction, as any side of an octagonal pile, for example, may become subject to tension.

Some designers add 100 percent to the weight of a pile to provide for the shock due to handling. This may be excessive in cases where special provision is made for proper handling, 50 percent being a more reasonable allowance. Very long piles have extra longitudinal reinforcement provided in the middle third or half of the length. Sometimes short lengths of additional longitudinal bars are inserted in the head, to aid in resisting the impact due to the hammer, but additional hooping is more frequently provided for this purpose.

The lateral reinforcement is of two distinct types: One consists of separate wire hoops or binders either approximately square or circular in shape, and spaced at intervals which vary more or less along the length of the pile; the other consists of a continuous spiral wrapping which varies in pitch at the head and

foot of the pile. It is primarily intended to increase the resistance of the concrete to longitudinal compression, but the latter form may also aid in resisting diagonal tension. The lateral reinforcement is equally as important as the longitudinal.

The percentage of steel in the section area of the pile varies considerably in practice, ranging from about 0.6 to 2.8 percent. In the pre-molded piles for the approach of the Municipal bridge at St. Louis, the total reinforcement amounted to 1⅛ percent of the volume of the pile. Experiment has shown that hair cracks develop in handling when the reinforcement is less than 1 percent.

The section area of the head must be sufficient to support in direct compression the safe load for which the pile is designed. The safe unit-stresses to be adopted should depend upon the quality of the concrete, the percentage of reinforcement and its arrangement, as well as the character of the loading. If the pile is tapered, the critical section for direct compression is not at the butt or top of the head but at some distance below the surface of the ground.

The additional allowance for hard driving in any case is preferably made by a direct addition to the section area or by adding extra cement to the batch of concrete to be placed in the head of the pile. It is, of course, understood that when a pile acts as a column that it is to be designed as a column.

The New York City building code, as recommended by the National Board of Fire Underwriters, allows a maximum load of 25 tons per square foot of cross-section and an additional load of 6000 pounds per square inch of steel reinforcement in the section. The unit-stress on the concrete is therefore nearly 350 pounds per square inch.

In a given railroad pier foundation the piles were designed for a safe load of 50 tons. They are octagonal in form, the least diameter is 12½ inches and the reinforcement consists of four ¾-inch square bars. If in this case the compression on the steel bars be assumed as 10,000 pounds per square inch, the compression in the concrete is found to be 534 pounds per square inch.

As the design of reinforced-concrete piles is so comparatively new, engineering practice in regard to safe unit-stresses is not reduced to such narrow limits as in many other divisions of structural design. For a discussion of the principles involved and their application to illustrative examples the student is referred to standard textbooks on mechanics and on reinforced concrete.

The method of computing the lateral resistance of a pile is similar to that for the lateral resistance of a track spike as deduced in JACOBY'S Structural Details, Art. 8. In that case the maximum unit-compression on the wood is located at the upper surface but for a pile the bearing resistance of the upper strata of the ground may be less than that of the lower strata. Therefore, while using the same general method, the formulas given cannot usually be employed without some modification. It may be sufficient for practical purposes in many cases to assume the surface of the ground to be lowered more or less so as to make the resultant moment of the actual pressures equivalent to that of the theoretic pressure used in deducing the formula.

Under retaining walls where the piles receive a lateral thrust as well as a vertical load it is necessary to use reinforced piles to resist the flexure thus produced. The distribution of bending moments indicates that at least the upper part of the pile should have a uniform section.

ART. 47. CAST-IN-PLACE PILES

A cast-in-place pile is a concrete pile which is built in its permanent place in a hole prepared for the purpose. While only some types of the class of pre-molded piles are patented, all types of cast-in-place piles have been patented. The characteristic features of the latter class relate more specifically to the method of construction for each type and the appliances used for that purpose. In making the type known as the Raymond pile (see Art. 41) a tapering sheet-steel shell or casing is driven into the ground by means of a collapsible steel core

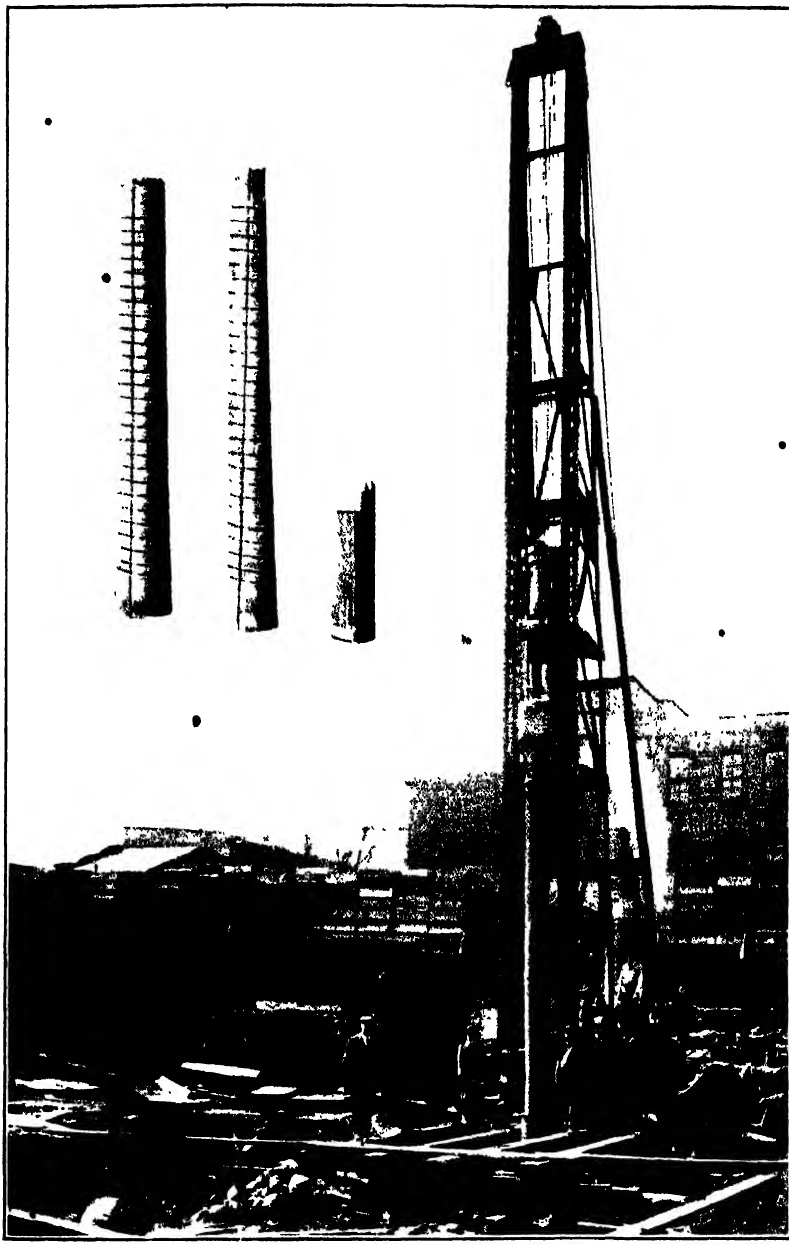


FIG. 47a.—Two Sections of Reinforced Sheet-steel Shell and "Boot" Section.
FIG. 47b.—Steel Pile Driver of the Raymond Concrete Pile Company.

(Facing p. 1420.)

which acts as a form to support the shell. After the desired penetration is reached the core is collapsed and withdrawn, and the casing filled with concrete. The core when dressed with the shell is driven by means of a pile-driver with a heavy steam-hammer. On account of the great weight of the core, the pile-driver is of heavy construction, steel leads and bracing being always used for the largest cores. The driver illustrated in Fig. 47*b* is equipped with leads measuring 57 feet from the top of the turntable I-beams to the head block. The shell of 18- to 20-gage sheet steel is made in various diameters and in conical sections about 8 feet long which overlap tightly (in telescope fashion) when in place, but enable them to be shipped knocked down, and to be readily slipped over the core in regular succession. The very short section closed at the bottom is called the boot, and is made of pressed steel to withstand the cutting effect of stone or other obstacles encountered in driving.

The object of the casing is to prevent the earth and water from mixing with the concrete and to act as a mold that shall preserve its shape until the concrete is set. Before placing the concrete, the interior of the shell can be inspected, by means of an electric light, by light reflected from a mirror or by the light reflected from the surface of water thrown into the casing.

More or less difficulty has been met when the hydrostatic pressure collapsed the thin shell, and sometimes several shells were driven inside of one another. In 1911 their construction was improved by reinforcing a 24-gage shell with a ¼-inch wire spiral, as illustrated in Fig. 47*a*, thus materially increasing its strength. The concrete is either a 1-2-4 or a 1-3-5 mixture, using respectively ¾- and 1¼-inch stone or gravel, and mixed rather wet.

The piles are occasionally reinforced by longitudinal bars but usually no such reinforcement is employed, unless short rods are inserted to assist in bonding the tops of the piles to the concrete footing. Lateral reinforcement is, however, provided by the spiral wire used to stiffen the casing, since the concrete of the finished pile is wrapped by it. Since reinforcement is seldom used in these piles, it is easy to place the concrete in the smooth shell so as to obtain good concrete without voids.

The standard sizes of Raymond piles have a diameter of 20 inches at the head for lengths of 20 to 30 feet, and 18 inches for lengths of 35 to 40 feet. The tip has a diameter of 6 inches for a length of 20 feet and 8 inches for greater lengths.

This type of pile has been very extensively employed in America, especially for the foundations of buildings. The special advantage claimed for it over those of other concrete piles are speed of placement, and economy due to the large taper (see Art. 55 for discussion of taper), whereby the length is materially reduced. The taper adopted is greater than that for any other type of pile. Additional advantages over those of other types of cast-in-place piles are the inspection of the form in which the concrete is deposited, and the testing of bearing power for every pile by the average penetration of the steel core under the final blows of the hammer. On account of using a steel core, it is claimed to be possible to drive through very hard material which cannot be penetrated by any other kind of pile at reasonable cost. Occasionally, a steel core is broken in such material as compact earth containing boulders.

The Simplex pile introduced in 1903 is made by driving a steel pipe, with a special shoe or "jaw" to close the bottom, in the same manner as a pile, and then filling the hole with concrete as the pipe is gradually withdrawn. The pipe must be extra-heavy and at least as long as the pile to be formed, and the pile-driver must have extra strength equipment to pull out the pipe. Sometimes a cast-iron or concrete shoe is used with a projection which fits into the pipe. The shoe remains in place and hence a new one is needed for each pile. Where the earth is firm and compact, an "alligator jaw" attached to the pipe by cable hinges is used which opens automatically when the pipe is withdrawn to permit the concrete to flow through it. A ram is generally employed to force each batch of concrete into place against the surrounding earth until the hole is completely filled; this increases the diameter of the pile somewhat beyond that of the pipe driven. In some cases the pipe is first filled with concrete and then slowly withdrawn at a uniform rate, without ramming the concrete. The concrete is made of a fairly wet

1-2-4 mixture using ¾-inch stone or gravel, which by its weight is expected to resist the pressure of the soil.

It is claimed, since the concrete is forced into the surface irregularities of the compressed earth, that its frictional resistance is greater than for any other kind of pile of equal diameter and length. The indentations, however become filled with compressed earth and become a part of the pile, thus changing the frictional or shearing surface to a more regular form. The concrete may also adhere to some stone or gravel contained in the surrounding material.

Where the earth penetrated does not have sufficient stability to retain its form when the pipe is withdrawn, this method cannot be used without modification. Such a condition has been met by dropping into the hole, after the first batch of concrete was placed, an auxiliary cylindrical form of sheet metal of slightly smaller diameter than the pipe. After this form is filled with concrete, the pipe is withdrawn. This leaves some voids outside of the sheet-metal form which will only be filled by adjustment of the surrounding earth.

As an illustration of the time saved in construction, it is noted that about 4800 Simplex piles from 30 to 45 feet long and aggregating about 162,000 linear feet were driven through filled material for the Terminal Warehouse Building at Pittsburgh in 76 days by seven pile-drivers. In another location piles 48 feet long were used. The practical limit to the length is the strength of the equipment provided to pull out the pipe.

It is impossible to inspect the integrity of the pile, and it is a question as to what extent its strength may be reduced by some admixture of the concrete with adjacent earth. In stiff, non-water-bearing, or clay soils, where the ground has no tendency to flow, this is claimed to be the cheapest system of installing concrete piles.

The pedestal pile invented by Hunley Abbott may be regarded as a modification of the Simplex pile by the addition of a bulb-shaped base or pedestal at the foot. Its form is intended to take a larger measure of advantage of a lower stratum of higher-bearing capacity, than is done by piles of the ordinary

form. By thus increasing its bearing area at the foot, it imitates the metallic disk and screw piles (Art. 59) which doubtless suggested it.

The pedestal pile requires the same equipment as the Simplex pile, except the shoe or jaw, and in addition a steel core which fits inside of the pipe with its enlarged head engaging the top of the pipe, and its lower pointed end projecting several feet below the pipe. As illustrated in Fig. 47*c*, the steel pipe and core are first driven into the ground and a charge of concrete dumped into the pipe. The core is next used as a rammer to enlarge the hole below the pipe laterally by pushing aside the concrete,

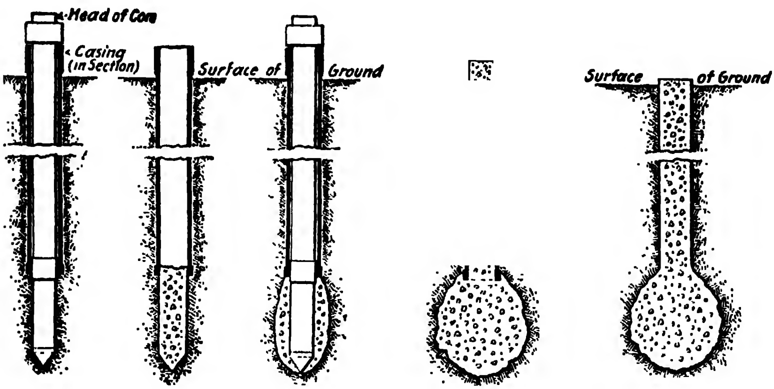


FIG. 47*c*.—The Process of Forming Pedestal Concrete Piles.

repeating the process until the concrete base has the required volume. Finally, the pipe is filled with concrete and then withdrawn.

The pipe employed is usually 16 inches in diameter and ⅜ inch thick, while the core projects 4 or 5 feet below the pipe. The cylindrical stem of the pile is hence about 17 inches in diameter and the base roughly 3 feet in diameter, the volume of the base being about 16 cubic feet. The diameter of the base, for a given volume of concrete used in making it, depends upon the nature of the ground and its homogeneity. If for any reason the earth should resist unequally on opposite sides of

the hole, the resulting form of base would make its reaction eccentric. The concrete is usually a 1-2-4 mixture, with the broken stone or gravel limited to a diameter of 1½ inches.

The pedestal piles under the retaining wall of the Oregon-Washington Railroad and Navigation Company at Seattle, Wash., were reinforced by six ⅝-inch rods 15 feet long, in order to provide against bending moments due to the horizontal component of the earth pressure.

Cast-in-place piles, except the Raymond type, cannot project any distance above the ground without the use of special forms at increased cost.

The Gow and Palmer pile is made by driving a metal casing or pipe into the earth, the inside of which is kept empty by a stream of water under pressure until it reaches the required depth. The casing is then withdrawn a few feet and a lozenge-shaped cutter lowered to the bottom of the hole. By turning this tool and at the same time opening it gradually the chamber is hollowed out and the earth removed by the current of water. The casing and chamber are then pumped out, filled with concrete and tamped, the casing being gradually withdrawn as the concrete fills the hole. Reinforcing bars are pushed down into the concrete to reinforce the stem of the pile when desired. This pile was originally designed in 1904 to underpin a building, the enlarged base being located in the clay stratum, which was overlaid by filling and soft material to a depth of 20 feet. The casing was in that instance put down in 5-foot lengths.

In one location where the original surface of stiff clay had been covered by 15 feet of clay fill from adjoining excavations, it was deemed best to carry the load to the underlying stratum. For this purpose holes 10 inches in diameter were excavated by means of a post-hole auger and then filled with concrete, the clay sides being stiff enough to retain the form of the holes. The piles were spaced 3 feet between centers, and the cost of the pile, was 33 cents per linear foot, 60 percent of which represented the cost of digging the holes. This extremely low cost was due in part to the absence of any charge for the installation of plant.

The chief objection to all cast-in-place piles has been based upon the probability of injury to the green concrete by driving the forms for adjacent piles. There are other disadvantages which pertain only to certain types. The precautions which may be adopted to obviate these difficulties are discussed in the next article.

ART. 48. PRECAUTIONS AGAINST INJURY

Since pre-molded piles cannot be driven until they are sufficiently seasoned, they may be placed in any order in the required foundation. This cannot safely be done with cast-in-place piles. When the core or the pipe is driven for a given pile, it displaces and compresses the earth adjacent to the hole which is formed, and the elastic earth tends to relieve its stress by crowding back. Even if the shell, which is left in the hole, or the weight of the concrete, when no shell is employed, is able to resist this outside pressure until the cement is set, it is very probable that the green concrete will be injured by the vibration and additional earth pressure due to driving adjacent piles, after the setting of the cement has progressed to a certain extent and before its completion.

To determine its effect, two tests were made before beginning the pile work in the foundations for the north abutment of the Pittsburgh and Lake Erie Railroad bridge over the Ohio River at Beaver, Pa. The first test consisted of a pile driven with four others around it, spaced as in the proposed foundation work, the four being driven while the test pile was still soft. The second test differed from the first only by allowing the test pile to set partially before the four piles around it were driven. The working conditions on a large foundation are such that the second test more nearly represents the actual conditions than the first. After both test piles were allowed 30 days to set, the first test pile supported a load of 60 tons for 72 hours with a settlement of 9/64 inch, which was recovered almost wholly after the load was removed, while in the other case, the results, as expected, were not good enough for approval. To meet the

difficulty developed by the conditions of the second test, and which apply to all kinds of concrete piles formed in place, thus recognizing the well-known limitations of concrete, the following specifications were adopted in 1908, probably for the first time on such work, and were strictly observed:

The setting of the concrete in any pile must not, under any consideration, be disturbed by driving another pile or piles within a radius less than 9 feet from it, center to center, after a minimum interval of three hours or before the expiration of seven days from the time the concrete was mixed with water for that pile. The contractor may, however, at his own option drive pile forms within the 9-foot radius to a depth not more than 3 feet from the total estimated penetration, inside of the three-hour limit; and then after the three-hour limit and before the expiration of the seven day limit, complete the driving and filling of these forms.

The extent to which such damage may occur has been proved by subsequent excavation in a number of cases, owing to changes in plan or to building adjacent structures. In one example, failure was due to the fluid alluvial soil penetrating between batches of concrete, thus separating the pile into sections about 5 feet long. In another, the cement failed to set on account of certain chemical constituents in the ground water, ascertained later by analysis. In still other cases, piles had their section areas reduced from 20 to 100 percent, and were bent out of line.

The liability of the green concrete to suffer injury by driving adjacent piles is increased when thin, hard strata alternate with soft ones. Material which is lighter than concrete may transmit pressures which displace concrete when it is soft, and injure it after the initial set. In boulders or gravel, a shearing effect may be produced instead of merely a direct pressure. Unless protected by a shell, there is more or less danger of some of the cement being washed out by underground flowing water, or, on the other hand, that the cement may be deprived of some of the water which it needs to set completely, by the absorbent earth.

It should be added that the construction of cast-in-place piles requires more careful supervision to secure good results on

account of the manner in which the concrete is deposited, and the surrounding conditions which preclude inspection of the pile after the concrete is all in place. When it is deemed necessary to put reinforcement in a cast-in-place pile throughout its length, it should be fabricated as a unit and properly put in position. It is impracticable to place bars separately, so that they shall occupy specified positions in the finished pile.

ART. 49. COMPOSITE TYPES AND COMBINATION PILES

Many concrete structures in sea water have failed because of the disintegration of the concrete. Whether this action is a chemical or physical one has not been definitely established, but it is probably a combination of the two. Among the possible causes of failure are the marine rock borers of the Pholad family, which have recently been found in Los Angeles Harbor in the mortar encasement of timber piles, but not in concrete containing foreign stone or gravel. These borers resemble the clam, being about 1 inch in diameter and from 2 to 3 inches long. Some engineers recommend painting concrete piles with an asphalt paint above low tide.

The denser the concrete the less the tendency to disintegrate. Piles have been built of "gunite," a concrete formed by the use of the cement gun. This concrete is much denser and stronger when properly made than ordinary concrete. In 1921 hollow piles built up by the cement-gun process were used in Los Angeles Harbor. The piles were circular in shape with diameters of 18 and 20 inches and lengths of 40 to 60 feet. The shell, made of 1-1½-2½ concrete, with a maximum size of aggregate of ½ inch, was 4 and 4½ inches thick. The reinforcement consisted of eight ⅝-inch vertical rods encircled by a heavy wire-mesh cylinder. A conical point of solid concrete was first cast and the rest of the pile then built in a vertical position on this.

In 1922 a patent was issued covering the treatment of concrete piles with a hot bath of asphalt, resulting in a type of pile called "Duocrete." This process consists of ramming a rather

porous mix of concrete—usually 1-3-3—in horizontal molds, and after 30 days immersing the piles for 24 hours in an asphalt bath at a temperature of 500 degrees Fahrenheit. After cooling the bath to a temperature of 212 degrees the asphalt fills all cavities developed by the condensation of the steam. By using a 3-inch, tapered, removable core it is possible to penetrate all positions of the concrete. Recent experiments indicate that by using the alternate vacuum and pressure treatment the temperature can be kept down to 300 degrees Fahrenheit. "Duocrete" is stronger and tougher than ordinary concrete, but not enough time has yet elapsed to determine its lasting qualities in sea water.

Hollow pre-molded piles, which were filled with lean concrete after they were placed in their final positions, were driven in 1911, for the foundations of the ocean pier at Long Branch, N. J. The piles are of the Hennebique type and range in length from 45 to 68 feet, with an average penetration of 22 feet. Near the shore, some 18-inch square piles were used, but the rest are hollow square piles 24 inches square on the outside and 13 inches on the inside, in order to reduce their weight for handling. The reinforcement consists of 1¼-inch round rods tied together at intervals with ¼-inch wire collars, while the longer piles have additional reinforcement in the middle to provide against breakage by handling. They were handled by a special form of sling or bridle, to reduce the stresses due to bending.

The largest hollow, concrete piles on record, 39 inches in diameter and 149 feet long, were used for the substructure of a bridge in Stockholm harbor, the piers consisting of reinforced concrete caps, carried by groups of piles in a manner similar to timber-trestle construction. The pile shells were 3 inches thick and well reinforced. The concrete was a 1-1¾-1¾ mix, with a strength of 3500 pounds per square inch in 28 days. After the piles were driven they were cleaned out with an air ejector and filled with concrete.

The "peerless" concrete pile has a sectional reinforced-concrete shell, which is driven down together with a steel driving pipe, both of which bear on a pointed cast-iron shoe, which is

left in the ground. After the steel pipe is withdrawn, the shell is inspected, and filled with concrete by a special tremie designed for the purpose. The use of the steel pipe protects the concrete shell from severe stresses due to driving.

Reinforced-concrete piles, with a diameter of 25 inches, were placed under the Music Hall at the reconstructed pier in Atlantic City in 1906. Since the largest of these piles were nearly 50 feet long, it was deemed impracticable to mold them complete before sinking them in place. Accordingly, the lower portion, 12 feet long, which included an enlarged foot, 3½ feet in diameter and 2 feet high, was molded in a wooden form with the jet pipe and steel reinforcement in place. When the concrete was hardened, a 3/16-inch galvanized steel shell was slipped a little distance over its top, and the joint made water-tight by calking with oakum. The steel shell was made water-tight by close riveting and calking, and was long enough to reach above the water when sunk. After the reinforcement of the upper part of the pile was hooked on and the jet pipe extended, the pre-molded pile and casing were swung into place and sunk about 16 feet in the sand by the water-jet. The steel form was then filled with concrete.

Where the conditions are such that the water-jet cannot be used to sink them and there is danger of damage to pre-molded piles by driving, the following method may be adopted: A steel shell and cast-iron shoe are driven to the proper penetration by the method used for Simplex piles; some concrete is then placed in the shell and a molded reinforced-concrete pile is inserted and embedded firmly in the concrete in the bottom of the hole, after which, the intervening space is filled with a strong grout and the shell is withdrawn. The quantity of grout used is to provide some excess over that required to fill the space between the molded pile and the sides of the hole after the shell is withdrawn.

This method was specified for the pile foundations of a reinforced-concrete structure of the Pittsburgh and Lake Erie Railroad at Youngstown, Ohio. The molded piles were octagonal in form, 12½ inches thick, of uniform cross-section

throughout, and the lengths were determined by driving test piles. The molded piles were varied in length by steps not exceeding 5 feet and generally smaller; and the top of the moulded pile was brought to the proper elevation by adjusting the quantity of concrete filling, which ranged in depth from 2 to 5 feet after the shell was withdrawn. The reinforcing rods extended 2 feet below the bottom of the molded pile.

It was specified that the Simplex forms and shoes were to be driven to a penetration of ½ inch per blow from a 3000-pound hammer, falling 15 feet. No inserted pile was to be exposed to stresses due to driving adjacent piles, until it had been in place 15 days. The piles were designed to carry 50 tons per pile safely, and those selected for testing were to settle not more than a half inch under a load of 60 tons. It was estimated that the use of molded piles thus inserted, at an increased cost per pile, would reduce the total cost of the foundation by diminishing the sizes of the pier footings, under the assumption that their safe bearing power is 25 percent greater than that of cast-in-place piles.

On the Pacific Coast, combination piles have been made for wharf construction by driving a wooden pile from 50 to 60 feet long with its head projecting 10 feet above the mud line, which is 20 or 30 feet below the top of the wharf. A hollow reinforced-concrete pile 2 to 3 inches thick and 24 inches in diameter is then driven over the wooden pile to a good bearing in the mud. After removing the mud and water inside, the hollow pile is filled with concrete. Such combination piles can also be used in foundations on land, and are considerably cheaper than very long concrete piles.

Similar combination piles were used in the Delaware Lackawanna and Western Railroad Terminal at Hoboken, N. J., but in this case the form for the concrete top was attached to the pile and carried down with it into position so as to avoid the necessity of pumping out the form. The forms were filled after the follower was withdrawn.

Occasionally, the durability of concrete piles is combined with the lesser weight and cost of timber piles, in a single building

foundation by placing concrete piles on top of timber piles, which are driven below ground-water level with the aid of a follower. In case the concrete pile is built in place, a waterproof tube or container is put on top of the timber pile, filled with concrete, and after the concrete is hardened sufficiently, the upper end of the container is cut away at the lower surface of the concrete cap or footing.

Figure 49*a* illustrates a patented composite timber and concrete pile. A mandrel first drives a 16-inch steel shell, through

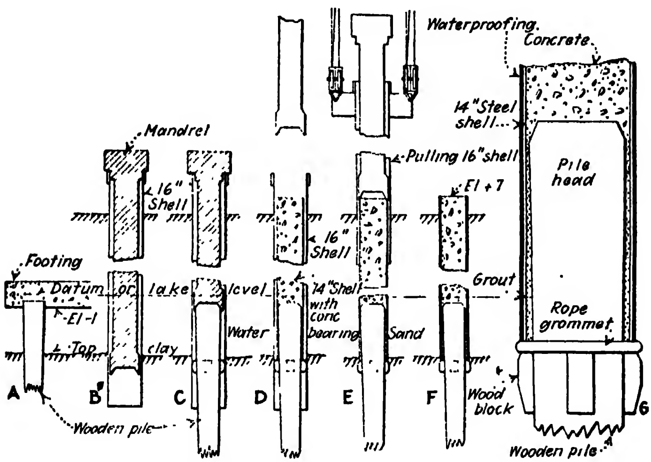


FIG. 49*a*.—Composite Timber and Concrete Pile.

which a timber pile is then driven. A 14-inch steel shell, protected outside with asphalt membrane waterproofing, is then forced down over the top of the timber pile for a distance of 30 inches. Grout is next poured into the 14-inch shell to fill the space around the head of the pile, after which the 16-inch pipe is withdrawn and the 14-inch steel shell filled with concrete.

The Ripley combination pile, shown in Fig. 49*b*, is composed of a timber pile encased in concrete. The reinforcement consists of wire mesh wound spirally with the concrete around the pile, to which it is attached by staples, the final lap being tied with wire. Before concreting, spikes are driven into the timber

at intervals on its surface. The concrete is a 1-2-3 mixture of cement, sand and broken stone.

Another combination pile is made by shooting a 2-inch thickness of "gunite" around timber piles as noted in Art. 24. Care must be taken in placing the concrete encasement to have the timber thoroughly water-soaked, else the subsequent swelling will break the concrete. The "gunite" is usually placed in two layers with a reinforcement consisting of longitudinal bars and wire mesh.

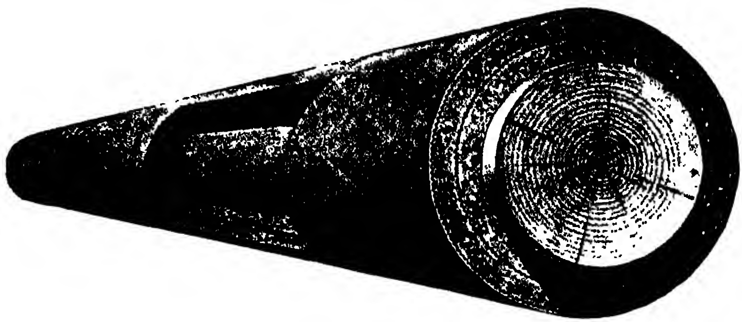


Fig. 49b. Ripley Combination Pile.

Art. 50. Drivers, Hammers and Caps

To drive pre-molded piles the pile-driver and its equipment have to be strong on account of the greater weight to be handled, and the heavier hammers used. Piles weighing from 2 to 4 tons are quite common and those of 6 to 8 tons are employed on heavy construction, while the record weight is 32 tons for a 24- by 24-inch pile, 110 feet long, used in 1924 for Pier 7 of the Port of Manila. On this account the steel pile-driver is growing in favor. It is found to be stiffer, more durable and lighter for the same strength than those built of wood. The necessity for dragging the piles from the casting platform or from the unloading platform to the driver develops stresses in the tower for which special provision must be made in the design.

Concrete piles should be driven wherever possible with the aid of the water-jet, so that the duty of the hammer becomes

secondary. However, in some kinds of earth it is necessary for the hammer to do very effective work either with or without the aid of the water-jet. Under such conditions it is uneconomical to use a light hammer which may answer very well for a timber pile but which itself is considerably lighter than a concrete pile. Otherwise the temptation is constantly present to use too high a fall and thus expend too large a part of the energy in useless or destructive work.

In driving concrete piles into hard clay for the foundations of the Kentucky and Indiana bridge to which reference was made in Art. 45, a steam-hammer was at first used in which the striking parts weighed 3000 pounds. Upon substituting another one with a 6000-pound striking weight, the results were far more satisfactory.

A high-speed hammer will often shatter piles more than a slow-speed one. In driving piles in California a 17,500-foot-pound energy hammer working at 56 blows per minute gave much better results than a 10,150-foot-pound energy hammer at 110 blows per minute.

Although drop-hammers weighing less than 4000 or 5000 pounds have been employed to drive concrete piles successfully, the time required was unnecessarily large to secure the required total penetration. Very satisfactory results have been secured on some building foundations in Pittsburgh by using hammers weighing from 7000 to 12,000 pounds each. Such hammers are handled by three-part, crucible-steel lines rove at the lower end over sheaves set in the hammer casting. The fall of the largest hammer is limited to about 8 feet, but is usually less, and it has been used to drive concrete piles weighing about 3000 pounds to an average depth of 30 feet below the surface with a penetration in the gravel of 1 inch for the last 10 blows of the hammer. Three machines, operated by a total crew of 25 men, have averaged 15 piles per day for each machine, with a maximum of 25 piles, while driving through mud and clay which overlay a deep gravel stratum.

The heaviest steam-hammer built prior to 1925 was a Union make with a total weight, including base, of 28,000 pounds and

with striking parts weighing 4000 pounds. Its length of stroke was 36 inches. It was especially designed to drive concrete piles 24 inches square and 47 to 77 feet long for the monolithic concrete piers, docks and breakwaters of the Canadian government at Halifax, N. S. A hammer of the same make, but weighing, with follower casing, 30,000 pounds was used in driving the 32-ton piles noted in the first paragraph of this article. A single-acting hammer has been built with moving parts weighing 7500 pounds, and a total weight of 16,250 pounds.

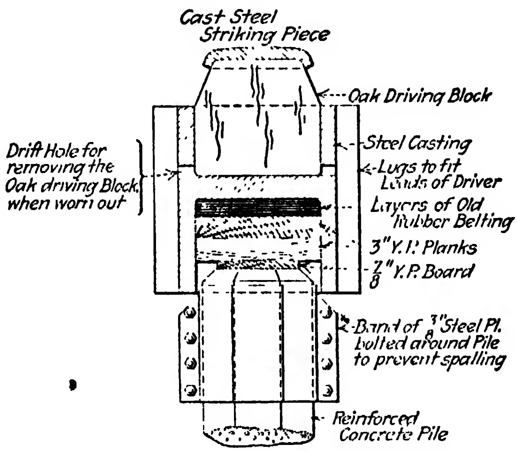


FIG. 50a. - Cap for Driving Concrete Piles.

Those who have had experience with both steam- and drop-hammers in driving concrete piles state that the steam-hammer drives them in less time and with less injury to the pile. Excellent results have, however, been obtained with the drop-hammer, the heavier hammers being the more efficient.

The successful driving of pre-molded piles without injury, when it is necessary to use the hammer actively, is due mainly to the various driving caps which have been devised as the result of experience. Figure 50*a* shows the form used for the foundation piles of the Municipal bridge approach at St. Louis. The construction is fully explained on the diagram. It was found that as long as the blow could be uniformly distributed

on the pile head but little injury was produced, but when the pressure was too large on the edges the pile head spalled and exposed the reinforcing bars. Hence in some cases the head had to be cut off in order to continue driving.

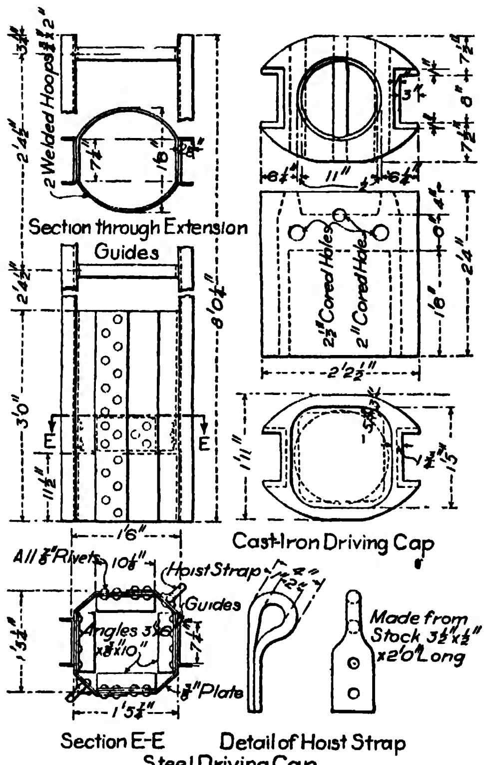


Fig. 50*b*.—Steel and Cast-Iron Shells of Pile Caps.

Figure 50*b* gives the details of the shells of two driving caps which have been used on the Chicago, Burlington and Quincy Railroad. The steel cap which was first used consists of a built-up shell of 3/8-inch plate, with steel channel guides riveted to the sides. The extension guides are to keep the cap in the leads when driving below the track, thus avoiding delay in reentering it. Near the lower end of the shell is fitted a 3-inch oak block and above that a 6-inch layer of rope ends, old rubber

hose or belting. Resting on top of this cushion is a short piece of wooden pile which extends above the guides and receives the blows of the hammer. For use with the cast-iron cap, the cushion consists of two layers of old rope or a bag of sawdust. After the pile is placed in position the cushion is laid on top of its head and the cap lowered over it. The top of the cap contains a short driving block. As shown in the plan, this cap is designed to fit either a round or a square pile. Its jaws are chamfered at the top to facilitate reentering the leads when it is driven below them. The cast-iron cap was found to be more satisfactory in service than the steel cap in which the rivets holding the angles broke repeatedly. The cap used in pile driving at the Kentucky and Indiana bridge at Louisville was square in cross-section, and composed of two bent steel plates bolted together through the projecting flanges at the sides. On the other two sides channels to engage the leads were riveted by means of two pairs of intermediate horizontal Z-bars. After a number of experiments with cushions the best results were obtained by placing three cement bags filled with coarse sawdust directly on the head of the pile which projected several inches up into the steel cap. A square block of beechwood 2 feet long was placed on top to receive the blows of the hammer. This species of wood proved better than any other. In some cases where fine sawdust was used the pile heads shattered under very hard driving.

At the Cleveland docks of the Pennsylvania Lines (see Art. 43) a cast-iron cap was used with an oak filler block on top and a few coils of rope underneath. At Cambridge (see Art. 51) the steel-plate cap was 16 inches square on the inside and inclosed an oak block 18 inches high, to the bottom of which six thicknesses of rope and four layers of rubber belting were nailed. The cap was held in the leads by two pairs of vertical oak pieces bolted through the incased driving block. At Brunswick (see Art. 43) a cast-steel cap was used with rope and rubber below and a wooden driving block above. The cap was made to fit over the tenon cast on the head and performed the important additional function of preventing the pile from turn-

ing while it was being driven. At the Chicago and Northwestern Railway bridge at Racine, Wis., the cast-steel cap was 3 feet high and had a solid horizontal diaphragm in the middle. The underside fitted the pile and rested directly on its head. On top of the diaphragm was placed a rubber cushion and driving block.

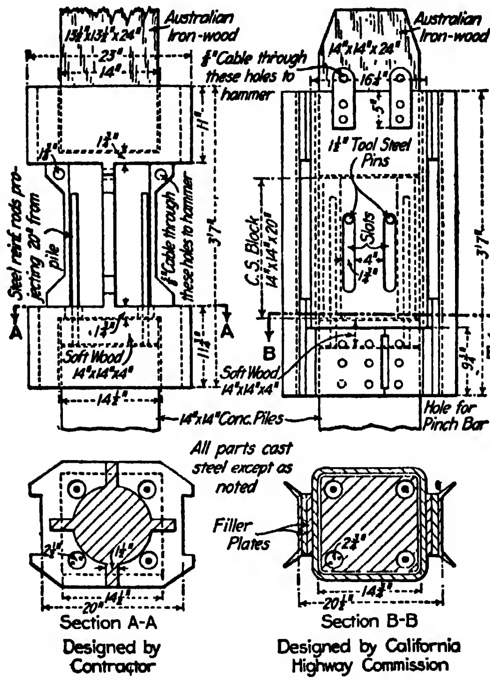


FIG. 50c.—Pile Caps Used by California Highway Commission.

In another case the cushion in the casting is composed of coarse sawdust or planing-mill shavings, above which rested a hard-gum driving block hooped with a steel ring. The sawdust or shavings are quickly compressed to adhere to the casting and only need occasional renewal. The driving block proved to be very durable. Sometimes rubber-lined canvas hose is combined with rope to form a cushion. In still another example, a mat of six layers of rope was placed on the pile head

below the diaphragm of the cast-iron cap, while sawdust and a hooped driving block were placed above it. Australian pine has also been employed for driving blocks.

Figure 50*c* illustrates two types of caps used very successfully where rods extend up from the pile.

In driving piles with the 28,000-pound hammer previously noted three separate cushions were used between the head of the pile and the ram of the hammer. On top the pile was placed 3 inches of spruce plank and on this rested a cast-steel follower about 4 feet in height designed to protect the projecting reinforcing rods. This follower consisted of a hollow steel cylinder with top and bottom flanges. The bottom flange was flat and the top flange had a depression on its upper face to hold a wood block about 15 inches thick bound with steel bands.

Figure 43*g* shows a cushion consisting of a 4-foot thickness of timber.

ART. 51. DRIVING CONCRETE PILES

Some of the difficulties encountered in keeping the pipe from clogging when the jet pipe is cast in the pile are due to improper construction of the nozzle. In ground containing a large proportion of sand the sinking can be done mainly by the use of the jet, the hammer serving merely as a weight or to give occasional light blows. Where clay is the predominant constituent of the ground, the nozzle clogs frequently when the hammer is actively employed. This difficulty can be overcome most readily by extending the diameter of the end of the nozzle back for 12 inches or more, thus substituting a short-pipe tip for a conical tip. For concrete piles it is generally preferable, however, to use two jets on the outside of the pile. The equipment, methods and precautions for the use of the water-jet described in Art. 18 apply likewise to pre-molded concrete piles. The use of the water-jet with adequate equipment in sinking concrete piles whenever subsurface conditions permit is to be urged not merely on account of avoiding any possible injury to the pile by driving with a hammer, but to save time and energy.

At Brunswick, Ga., after failing to make satisfactory progress by means of the jet and hammer, a new scheme was adopted, in which the pile itself was used as a hammer. A wire bridle was fastened near the top of the pile by which to lift it, but the cap and hammer were allowed to remain on top to give additional weight. The pile was raised from 18 to 24 inches and dropped, and while this process was continued the jet was constantly operated, until the driving was nearly completed. By this means the number of piles driven per day was increased to 6 fold. The water was then shut off and the last few inches driven by the hammer alone. This distance was increased to 8 or 10 inches in good clear sand, as the jet excavated deeper below the foot than in the other material. The ground penetrated varied from clear sand to hard clay. It was also necessary to penetrate a 2-foot stratum of soft rock composed of shells, sand and lime which was harder than any coral. About 50 percent of the 6000 piles had to be driven through that material. Upon taking up some piles driven through it the edges at the foot were found to be but slightly rounded off. The jet pipe was not reduced in diameter at the end, and the pipe did not clog from driving more than once or twice. Their dimensions are given in the eighth paragraph of Art. 43.

At the Charleston, S. C., pier the bottom consisted of marl containing a yellow clay and about 15 percent of sand, making it extremely hard and sticky. When exposed to the air for a few hours it required a hammer to break it. Some of the piles had to be driven through 38 feet of the marl, and this was accomplished entirely by the churning process until within 2 or 3 inches of the full penetration when they were driven to grade by a 4500-pound hammer.

In driving 800 30-foot concrete piles for the Sixth Street Viaduct in Kansas City, Mo., a hole was jetted down the full length of the pile in the proper position. The pile was then inserted in the hole and churned up and down with the hammer resting on top, while the jet was used alongside of the pile. Experience showed that two jets would have been better to secure sinking accurately in position. When the hole was not

first jetted down, the piles had a tendency to crowd toward those previously driven, since the ground on that side was still soft.

At Cambridge, Mass., where a 4700-pound drop-hammer and water-jet were used in driving piles for a building foundation, it was found best to begin driving by churning and the water-jet, and after continuing this method as long as possible the chain which connected the pile to the hammer during the churning operation was disconnected and the hammer started with a drop of about 2½ to 4 feet, and increasing the fall as the driving became harder. Sometimes the churning process can be employed advantageously to start a pile where the leads are not long enough, or a short wooden pilot pile may be driven first and withdrawn, and the pile then churned up and down in the hole after directing a stream of water into it with the hose.

In loam and ordinary clay it was the practice on the Burlington Railroad, as reported in 1911, to put down two or three holes with the jet as close together as possible. The pile was then set in the leads and driven without further use of the jet. It was found that this method saves time besides reducing injury to the pile. In many cases a penetration was thus secured which could not have been reached by driving with the hammer alone.

In order to save a considerable length of pipe an arrangement is sometimes adopted of casting a jet pipe only in the lower portion of the pile, its upper end having a reversed curve and terminating outside of the pile. The outside pipe can then be connected to this and afterward removed. This connection may be located just above the ground level in a pile extending above the water surface.

It is remarkable how well pre-molded piles usually stand the pounding of the hammer where the jet cannot be used successfully. In one instance where piles were driven into hard clay for a bridge pier, after several piles were driven the clay became so compact that it required 5000 blows of the steam-hammer to drive some of them 20 feet. In the few piles which were broken the crushing extended only 18 inches below the top of the head.

At the approach to the Municipal bridge, where the driving was very hard, only eight out of 767 piles were broken, due to the cold weather retarding the setting of the concrete. Apart from this the injury to other piles was confined to some spalling at the heads, and that occurred mainly in piles made in the winter. In some cases, piles stood 40 to 80 blows per inch of penetration, but most of the heads were uninjured after sustaining more than 2000 blows of a steam-hammer.

Piles of the Cummings type were driven 60 feet into the ground on a bridge for the Norfolk and Western Railway. A drop-hammer weighing 12,000 pounds was used with a 6-foot fall. Although the driving was so hard that only two piles a day could be driven, the average penetration being ¾ inch per blow, yet only a very small number of piles were damaged in driving.

Reinforced-concrete piles made in cold weather and imperfectly set, due to the cold, can be driven practically without fracture at low temperatures, or about 10 to 15 degrees Fahrenheit. When, however, the temperature rises above the freezing point, such piles will go to pieces under the hammer. But after the piles are thoroughly cured they can be driven without danger of fracture. In other words, in respect to driving, the effect of freezing is practically the same as that of thorough setting of the concrete.

In driving 675 concrete piles, molded vertically in steel forms, to a penetration of 20 to 30 feet for bridge piers in Cleveland, no failures occurred and no pile heads were battered. A 5-ton steam-hammer was used. Thoroughly well-seasoned concrete piles will stand without appreciable injury several hundred blows with a 3000-pound drop-hammer, the drop increasing from 10 to 30 feet as driving progresses, but comparatively green piles must be handled very carefully and the drop limited to 6 or 8 feet. Such work is slow and expensive, and it is better to season piles thoroughly.

In driving 3-ton piles under bridge abutments by the Sanitary District of Chicago, the time ranged from 9 to 27 minutes per pile for the driving, with 21 minutes or more to get the next

pile ready. The average number of blows was 600, and the maximum 1782. One of the piles which required over 1600 blows was cut off 1½ feet, and no lines of weakness due to driving could be discovered. Of 300 octagonal piles driven by the Long Island Railroad on its Jamaica improvements not a single one was broken either in handling or in driving.

In 1907 a pre-molded pile 30 feet long, in which the reinforcement was electrically welded into a unit form, was selected at random from a thousand that had been driven for a dock pier. A careful examination of the pile after it was pulled up failed to reveal any defects. The same pile was thereupon driven and withdrawn twice in different locations through 20 feet of silt, sand and gravel into soft rock, without any sign of deterioration. Finally, it was driven again for permanent use in the pier.

At Cambridge in 1908 a reinforced-concrete pile struck a boulder at a depth of about 18 feet and could be driven no further. The 4700-pound hammer with drops of 18 to 30 inches had given it 735 blows, the water-jet being used also. As the head was badly crushed the driving was stopped; the projecting part was cut off, its ends squared, and sent to the Watertown Arsenal for test. Its length was 9¼ feet, its smaller section area 128.59 square inches and it developed a compressive strength of 3865 pounds per square inch. Since this value exceeds the usual strength of reinforced-concrete columns, the pile evidently suffered no injury due to hard driving except at the head which was cut off.

Another method has been used in hard clay which resisted penetration by the use of the steam-hammer except at too great a cost in time. The piles were 14 and 9 inches square at the butt and tip respectively, 22 feet long and driven to rock. Holes 12 inches in diameter were bored with a post-hole auger from 16 to 19 feet deep in which to place the pile and start driving. The weight of the hammer would push the pile down 8 to 11 feet. In driving the piles, great care was necessary to center the leads directly over the piles so as not to cause bending in the pile. Only one out of 125 piles was shattered enough to condemn it, and only three required new heads to be cast.

After gaining some experience, 90 percent could be driven without a crack, and in the balance the cracks were confined to the topmost 12 inches. It was found necessary to stop driving at intervals to permit the compressed air and water in the auger hole to escape through the gravel next to the rock.

In Art. 49 reference was made to a method which avoids driving the concrete pile itself, by first driving a very heavy steel tube fitted with a point or shoe. After it has penetrated to a good bearing, a few cubic feet of concrete are deposited in the bottom of the form. A pre-molded pile, slightly smaller in diameter than the tube, is then lowered to place and forced into the plastic concrete. After withdrawing the tube the remaining space is filled with grout. By this method a pile may be forced some distance into stiff clay or hardpan which is overlaid by soft material that would not otherwise hold the pile in place laterally.

Concrete piles cannot be driven as rapidly as timber piles on account of the care necessary in handling the greater weights, and the extra work in getting ready to drive, as well as the necessary delays incidental to driving. In one case where three crews were working on the same foundation, one drove 41 piles, aggregating 1207 linear feet, in 872 hours, another 39 piles, or 1130 linear feet, in 9 hours, and the third 45 piles, or 1064 linear feet, in 10 hours. In another case 42 piles were driven in 10 hours. No soil has been encountered in which wooden piles can be driven in which it has not been possible to drive concrete piles, and in many cases with far less danger of overdriving. At Greenville, N. J., a Chenoweth pile 13 inches in diameter and 50 feet long was driven into the ground and penetrated 8 feet into a substratum of gravel, and was subsequently withdrawn. A wooden pile could be driven only 2 feet into it.

Occasionally, it is found to be impossible to drive a concrete pile to the proposed depth, and it becomes necessary to cut off its head to a given grade to connect with a concrete footing. A track chisel and heavy hammer may be used for the concrete and a hack saw for the reinforcing bars. A plumber's pipe

cutter has also been used for round reinforcing rods. Since the concrete is usually not more than a month old when driven, the task of cutting it is not so difficult as for concrete which is thoroughly seasoned.

ART. 52. ANALYSIS OF TIME AND COST

In order to obtain data for estimates of cost for pile driving, a series of observations was made in 1908, by SANFORD E. THOMPSON and BENJAMIN FOX, of the time required for each elementary operation into which the process of pile driving was analyzed. The results in detail, together with the conclusions and some recommendations intended to facilitate pile-driving operations by better system and less waste of time, are published in the Journal of the Association of Engineering Societies, vol. 42, page 1, January, 1909.

The ground at the site, as was shown by explorations, consisted of 6 to 8 feet of fill; and then, to a depth of 29½ to 31½ feet from the surface, fine sand and mud—but which was practically considered all sand—underlaid by a clay hardpan which was tested to a depth of 13 feet. The piles were 14 and 9 inches square at the butt and tip, each one being reinforced by four ⅞-inch corrugated bars with loops of ¼-inch bars spaced about 12 inches apart but reduced to 4 inches near the head. Extra longitudinal reinforcement of ⅜- or ½-inch bars 2 or 3 feet long was also put in the head. A galvanized pipe was cast in the center of each pile for the water-jet. For experimental purposes the pipes were 2, 1½, 1¼ and 1 inch in diameter. The piles were seasoned from 30 to 41 days. A drop-hammer was used weighing 4700 pounds.

"After moving the pile-driver, the usual [work preliminary to the actual] driving consisted in hooking and dragging the pile; lifting it to place and attaching the hose, or attaching the hose first and then lifting; and setting the pile in the leads. The water was then turned on and the pile usually penetrated for a short distance without the hammer. The hammer was then placed on the cap and the pile sank further to a depth

depending upon the nature of the fill. Next, the hammer was attached to the pile with a chain and the churning commenced. There was enough play in the chain connection to give about a 10-inch blow of the hammer each time the pile was lifted. When this churning became ineffective the chain was disengaged and the pile was driven with blows in the usual manner."

The elementary unit-times were obtained in sufficient detail so that they may be recombined in any desired arrangement. "This enables the constants to be distinguished from the variables, abnormal times corrected, and lost time which will not occur on another job eliminated. Allowance can be readily made for the time which is always necessarily lost during rests and ordinary delays."

The average time per pile was found to be as follows: For moving the pile-driver, 29.0 minutes; placing the pile, 23.0 minutes (including delays 5.1 minutes); driving, 83.0 minutes (including delays 21.3 minutes); a total of 2 hours and 15 minutes. As the men became more expert in moving the driver and placing the piles, their average times were reduced in the last four days to 27 and 13 minutes respectively, the former, however, being still unnecessarily long on account of imperfect rolls under the driver. The time of driving was greatly increased by the low pressure of the water-jet. Taking an average for 16 piles driven in less than an hour each, the time during driving was 44 minutes, making the total 1 hour and 24 minutes. Expressed as percentages, the three operations require, respectively, 32.1, 15.5 and 52.4 percent of the total time. One-half of the delays were said to be avoidable.

A further analysis of the time required to get ready to drive, exclusive of delays, gives the following percentages: attaching the rope to the pile, 14.1; dragging the pile to the driver, 30.4; attaching hose and ropes preparatory to raising, 15.6; raising the pile to a vertical position, 12.2; placing the pile in the leads, 15.9; and placing the hammer and cap on the head of the pile, 11.8.

The number of blows of the hammer varies from 112 to 1160, the average being 589; the average range in the fall of

the hammer is from 1.0 to 5.6 feet, exclusive of one range from 25 to 20 feet; the average fall for the last blow is 4.7 feet, exclusive of one drop of 10 feet; while the average penetration under the last blow is closely ¼ inch, the maximum value being ⅞ inch. The total penetration varies from 25.6 to 32.0 feet, exclusive of one of 18 feet, the average being 28.7 feet.

The same investigators made the analysis of the cost for making and driving the piles, expressed in cents per linear foot of piling, which is given in the accompanying table.

Accordingly, the total cost per linear foot for making and driving the piles is $1.64. The cost for items 1 and 28 are based on the assumption that the plank is used four times. A few of the items, such as 12 and 13, are constant per pile and independent of the length, and may, therefore, be modified for a close estimate. The only items depending upon the character of the ground are 22, 24, 25 and 27. The cost for these items is based on the assumption of driving five and three-fourths piles in eight hours, and hence the corresponding cost can be estimated for a harder or softer ground by assuming the number of piles to be driven per day.

To make similar records of value for other estimates, the following elements must be kept in mind: (1) "To distinguish between the times which are constant for any job and those which vary with the quantity of the work; (2) to separate items which may be abnormally large or abnormally small on the job in question, so that allowance may be made for these particular items in future estimates; (3) to separate the time necessarily wasted because of abnormal conditions, or because the work is of a new or untried character."

Timber piles covered with "gunite" driven in 1922 in Tacoma cost the purchaser 15 cents per foot for piles peeled and delivered, $1.20 per foot for guniting and 21 cents per foot for driving and cutting off.

		Cents
1.	Plank for molding platform	2.56
2.	Lumber for chamfer	0.72
3.	Spikes for platform }	
4. 5.	Nails (9d. and 4d.) for forms }	0.11
6.	Crushed stone	5.12
7.	Sand	1.26
8.	Cement	8.64
9.	Longitudinal reinforcing bars	26.70
10.	Lateral reinforcing loops	4.01
11.	Wire to bind reinforcement together	0.50
12.	Extra-short bars in head	0.79
13.	Nipples for jet pipe	0.49
14.	Ells for jet pipe	0.39
15.	Jet pipe	3.46
16.	Hooks to handle pile	0.82
17.	Bending and placing reinforcement	8.38
18.	Labor on pile platform	2.26
19.	Labor on forms	5.72
20.	Labor on concrete	7.51
21.	Superintendence for making piles	2.13
22.	Pile-driving labor	27.22
23.	Cutting slot in tip of pile	0.20
24.	Repairs to pile-driver and cap	1.52
25.	Cutting off broken piles	1.61
26.	Rent of engine	2.07
27.	Superintendence for driving piles	2.86
	Cost varying with number and length of piles	117.05

28.	Plank for sides of forms	$17.50	
29.	Plank for ends of forms	7.50	
30.	Pile-driver, 25 percent of cost	49.55	
31.	Getting ready, two days	60.00	
32.	Teaming for pile-driver, etc.	34.55	
33.	Removing driver	34.61	
	Total cost for the job	$203.71	
	Cost of items which are constant for each job		13.91
	Total estimated net cost per linear foot if the job has 48 piles		130.96
	Add 25 percent for pumping, connections, contingencies and profit		32.74
			163.70

ART. 53. FORMULAS FOR BEARING POWER

In Art. 29 a reference is made to the relation between the weights of the hammer and pile. The formulas for the bearing power of piles given in Arts. 27, 28 and 32, except that of the Navy Department, do not take this into account by means of a separate term, but it is understood that this relation must be considered in any rational use of the formulas.

On account of the great weight of concrete piles this relation becomes one of increased importance, but it does not seem to be sufficiently appreciated in practice. Conservatism tends to employ the same weights of hammers for concrete piles as for timber piles, and to increase these weights for new equipment but slowly. Progress in this respect may be materially aided by the use of formulas in which the weights of the pile are introduced separately. Several formulas of this type are in extensive use in Europe.

EYTELWEIN'S formula in its ordinary form gives the ultimate resistance, but if one-sixth of its value be taken, as in the Engineering News formula, it becomes

$$\text{Safe load} = \frac{2W_hH}{s\left(1 + \frac{W_p}{W_h}\right)}, \tag{1}$$

in which the W_h denotes the weight of the hammer, W_p that of the pile, H the fall in feet and s the average final penetration in inches. It will be noted that if the penetration is 1 inch, and the hammer and pile have the same weight, that the value of the denominator is the same as if it were $s + 1$, but if the hammer weighs twice as much as the pile, the safe load is increased 33 percent. If the penetration be ½ inch, the bearing power by the Engineering News formula is $1.33\ W_hH$, whereas equation (1) gives $2W_hH$ and $2.67W_hH$, for the two cases when $W_h = W_p$ and $W_h = 2W_p$. These results indicate such radical differences that an urgent need is shown for careful comparative tests for driving concrete piles with different weights of hammers and within a limited range of fall.

RITTER's formula may be written in the following form:

$$\text{Ultimate load} = \frac{W_h}{W_h + W_p} \cdot \frac{12 W_h H}{s} + W_h + W_p, \qquad (2)$$

in which the terms are the same as those defined in the preceding paragraph. When $s = 1$ and $W_h = W_p$, one-sixth of the first term has the same value as the Engineering News formula, but when $W_h = 2W_p$ its value is increased 33 percent. This formula differs from EYTELWEIN's for the ultimate load merely in the added weights of hammer and pile.

As a result of extensive experience by the Raymond Concrete Co., in driving heavy collapsible cores for cast-in-place concrete piles, M. M. UPSON states that the Engineering News formula may be safely used to determine the approximate bearing power, and that the bearing power of the core may be applied to the cast-in-place pile, provided the compression of the soil is not released by the collapse of the shell. Steam-hammers are generally employed, the heaviest hammer being used for the longest standard core.

It has been truly said that no formula for pile driving can give more than an approximation to the supporting power of the special pile observed, and only at the time of driving; but with an intimate knowledge of the soil conditions, a good formula becomes valuable, and considerable money can often be saved by its proper application. In this manner the science of pile driving can influence the art. The peculiar and apparently erratic variations in the results obtained can be readily and satisfactorily explained by conditions in the ground, but they prove that it may be misleading to use a formula when no exploration has been made of the subsurface conditions at the site.

ART. 54. CHOICE OF TYPE

When it is determined in any given case that the use of concrete piles is justified by considerations of economy in which due allowance is made for durability, as well as the other elements referred to in Art. 43, and the conditions at the site

are known as the result of careful explorations of the ground (ART. 179), the next question is to decide what type of pile is especially adapted to these conditions, due consideration being given to the certainty of securing adequate strength at reasonable cost.

It may fairly be assumed that each type of pile has some distinctive advantages which are adapted, more or less closely, to certain conditions of the ground where piles are necessary. To use a type of pile, under conditions which are not favorable, involves either an economic loss, or a smaller degree of security, or both. Naturally, some types may be applicable to a wider range of conditions than others and it is the duty of the engineer to study each situation as a special problem.

When piles are used to support a structure above open water, as in pile trestles, wharves, piers, etc., they are required to resist flexure, as well as to act as columns. Pre-molded piles are the only ones which are adapted for this service; and they should be molded without taper, at least for that part of the length which is not in the ground. If the piles penetrate sand, which is not liable to scour, that portion may be tapered, since in sand the supporting power is almost wholly due to friction. If, however, the sand is liable to scour, or if adequate total penetration can be secured to furnish the necessary frictional surface, as well as the required horizontal reactions without exceeding the safe bearing value on the side of the pile, then a pile with uniform cross-section should be used.

In ordinary sand, quicksand, or in combinations of sand with gravel or clay, so as to produce a porous mass, in which the water-jet can be used successfully, the pre-molded pile has special advantages.

When a pile is driven through soft material to a hard substratum, so that it must act as a column, it must be reinforced, and hence frequently the pre-molded pile is the only type that can be used. The pile should be uniform throughout, so as to have a large bearing area in the harder substratum. Whether other types can be used depends upon the nature of the overlying material. If any stratum contains quicksand or other

soft material, which will not retain its form until the pressure of the concrete can resist the external pressure, then no cast-in-place pile should be used that does not leave a casing in the ground which can retain its form until the concrete has set. If such a shell or casing is used, it should also have uniform diameter, so as to secure a larger bearing area at the foot than that for a tapered pile.

If, however, the overlying material is of such a nature that it will retain its form temporarily, until the concrete is in place, then those types in which the pipe is gradually withdrawn may be used. If the underlying stratum which is to bear a considerable part of the load is not sharply defined on its upper surface, it may be desirable to increase the bearing surface of the pile by means of an enlarged base. The method of forming the pedestal pile requires the material adjacent to the base to be displaced by the pressure of the concrete due to ramming. If the material is not homogeneous, the base may be unsymmetrical about the vertical axis, and thereby produce an eccentric reaction on the pile column, thus causing dangerous stresses in the stem. In any case, when the load is mainly carried to its foot, the pile must be reinforced, unless the overlying material affords good lateral support, and there should also be some limiting ratio of length to diameter.

It should be remembered that if, subsequent to the installation of plain concrete piles, the adjacent ground is subjected to very heavy loading, that in some kinds of earth, like stiff clay, lateral pressure will be developed, thereby causing serious bending moments which piles without longitudinal reinforcement may be unable to resist safely.

If, for example, the substratum is hard clay and the foot of a pile of uniform section does not afford sufficient bearing area, then an elarged base may be formed by a tool like that referred to in Art. 47.

When the ground is compressible at the top but not soft, and gradually increases in density downward, any one of a number of different types may be employed, provided proper precautions are taken, but all of them should be without taper, so

that proper advantage be taken of the greater bearing at the foot and the greater frictional resistance of the lower surface of the pile. Pre-molded piles will probably require the use of the hammer, as well as the jet, or, if conditions on adjacent property do not permit the use of the jet, the driving may be done by the hammer alone. For cast-in-place piles, the necessary precautions relate more particularly to the order in which the piles are placed, so that no core or pipe is driven for another pile within a prescribed distance of any one during the setting of its cement (see Art. 48). When proper consideration is given to the importance of this matter, the relative cost of driving different types of piles assumes a different aspect. Usually, the economic relation will decide the choice of type of pile, and hence it is of the utmost importance that the same degree of security should be demanded for every one, so far as this is practicable.

When the ground near the surface is not quite sufficient to carry directly the load transmitted by a wall or column, with the aid of a spread footing, or when it costs less to increase its bearing power by means of piles, then the tapered pile of short length is most advantageous. Whether the pre-molded pile or one of the cast-in-place piles will be most advantageous, probably depends upon similar considerations to those described in the preceding paragraph.

If the ground consists of silt or alluvium for a great depth and increasing but slowly in density with the depth, so that the bearing power depends practically all on skin friction, the choice between a tapered and an untapered pile depends upon two factors. The pile with a uniform section has a slightly larger superficial area for a given volume, the greatest difference being practically less than 5 percent.' Such a pile has the additional advantage of having a larger proportion of its surface in the lower part of the pile, where the friction is slightly greater. But the tapered pile has a larger section area of concrete at the top to transmit the load and that may govern in some cases. As the load is gradually transferred to the surrounding earth in passing downward through the pile, the decreasing section area

of a tapered pile makes it conform more closely to one of uniform strength throughout.

If the ground is tough and leathery, so as to cause upheaval when adjacent piles are driven, it would be disastrous to use some types of cast-in-place piles; but so far as form is concerned the piles should not have any taper.

Sometimes deep beds of clay require pile foundations because the upper stratum becomes soft during the flood season, while during the most favorable time for construction, the clay is so hard that it is impracticable to drive any piles. Under such conditions, a satisfactory solution consists in excavating holes by means of an earth auger of the proper diameter, and then driving a pre-molded pile into it, so as to fill the hole so perfectly that the surface water will not follow down the pile.

Although some type of concrete pile may be adapted to nearly all kinds of earth, there are limitations imposed that leave a field of usefulness for the timber pile. Some black, marshy land will carry timber-pile trestles safely but nothing heavier than that. Concrete-pile trestles, with their reinforced-concrete caps and slabs, require the strata below the top to contain sand, gravel or stiff clay. In other cases, combination piles are used to reduce the load as well as the cost (see Art. 49).

Art. 55. Effect of Taper

To indicate the relative properties of tapered and straight concrete piles, the following example may be considered: Let the tapered piles be 20 feet long, and the diameters of its head and foot be 20 and 6 inches respectively, making its volume 20.2 cubic feet. Let a straight pile be taken having the same length and volume; its diameter is therefore 13.6 inches. In the tapered pile 44.5 percent of its volume is in the uppermost quarter of the pile, and 74.2 percent in its upper half; while 35.1 percent of its available surface for frictional resistance is in the top quarter, and 63.5 percent in the upper half of the pile. Since piles are frequently spaced 3 feet between centers,

let it be assumed that the compression of the earth surrounding a pile, which diminishes from the pile outward according to some law depending upon the nature of the material, be equivalent to a uniform compression, limited to a radius of 1.5 feet from the center of the pile. Dividing the depth into four quarters, the ratio of the displacement of the pile to the corresponding volume of the compressed earth is, accordingly, 25.8 percent for the top division, 16.9 and 9.8 percent for the next two divisions and 4.7 percent for the lowest division. For the straight pile the corresponding values are 14.3 percent for each division. The proportions of the total frictional area of the tapered pile are 35.1, 28.4, 21.6 and 14.9 percent in the four divisions respectively, while those for the straight pile are each 25.0 percent. The frictional areas of the tapered and straight piles are 68.1 and 71.2 square feet, the difference being a little less than 5 percent. It should be noted especially that about 45 percent of the total equivalent compression of the earth was expended in the top division, and very nearly 75 percent in the upper half of the depth.

It may be considered objectionable to adopt a large taper, since the compression of the earth is thereby made a maximum near the surface and a minimum near the foot of the pile, which is contrary to the fundamental principle of pile foundations, and since the area available for frictional resistance is reduced near the foot where the natural compression of the earth is generally the greatest and most useful. It should be added that the highly compressed and loaded area near the head of the pile may have its supporting power reduced by subsequent shallow excavations or by erosion in contiguous areas. Probably a more important objection to a large taper is that an increased bearing capacity is artificially created in the ground which becomes dissipated in time as the pressures are distributed through a larger mass. In districts subject to floods the bearing power of the ground near the surface is at least temporarily reduced, and if a large percentage of the load is carried by the ground near the surface serious settlement is very likely to result.

It should be remembered that in driving a straight pile the compression of the earth is done at the tip by increments as the penetration of the pile increases; on the other hand, in driving a pile with a large taper the compression thus made at the tip is materially smaller, but the compression is continuously increased all along the depth of penetration while the total resistance increases to its final maximum value. The tapered pile, however, causes less displacement or disturbance of the texture or internal arrangement of the material through which it is driven than the straight pile.

Experience in driving concrete piles into hard clay for the foundations of the Kentucky and Indiana bridge at Louisville, in 1911, led to a change in the taper by reducing the thickness of the head from 20 to 14 inches, leaving the thickness of the foot the same as before, or 9 inches, below which there was a pyramidal point 9 inches long. The piles were square in cross-section and 22 feet long. In some cases 5000 blows had been required previously for the 25-foot piles with the larger taper (see fourth paragraph of Art. 45).

Various tests have been made to determine the effect of taper upon the resistance of a pile. In a test at Chicago in 1901 a tapered steel core and an oak pile both 20 feet long were driven within a few feet of each other. The diameters of butt and tip were 18 and 6 inches for the core, 12½ and 10 inches for the oak pile. With a 2200-pound hammer falling 25 feet, the former penetrated an average of 1 inch for the last several blows, and the latter 5½ inches. The volume of the oak pile is 67.5 percent of that of the steel core.

In incompressible but plastic clays the wedge action of tapered piles is found to be of no value according to loading tests. Extensive experience proves, however, that concrete piles with a large taper have been used successfully in compressible ground to form foundations without subsequent appreciable settlement. In many cases, doubtless, the spread footing would have been a more appropriate type of foundation. In other cases, sand piles (Art. 60) might be preferable, for if the ground is to receive its greatest degree of compression

near the surface, it would apparently be a more economical arrangement to fill the conical holes made by the tapered core with sand, since sand is less expensive than concrete, and the increased bearing power of the ground could be utilized equally well by the concrete cap or footing (see Art. 154).

The following experiment is very instructive regarding the effect of taper. A concrete pile was driven to a total penetration of 26.5 feet, the diameters at the surface of the ground and at the foot being 18.6 and 8 inches respectively. The safe load was computed to be 40.9 tons. A wooden pile was driven to a total penetration of 24 feet, the diameters at the surface and at the foot being 11⅞ and 9½ inches. Its safe load was computed to be 11.6 tons. These piles were both driven in dense blue clay. They were subsequently loaded and the test loads for a settlement of ¼ inch in each case were 44 and 32.1 tons respectively. As the frictional surfaces are 92.4 and 67.2 square feet, the resistances are found to be 0.476 and 0.478 tons per square foot respectively.

No definite conclusion can be stated with respect to the effect of taper, since no adequate experimental investigation has been made of the subject. Tests are needed with piles of the same length and total penetration but with gradually increasing tapers, and these tests should be repeated in several typical kinds of earth. It is also desirable to have some sets in which the volume is constant, and others in which the frictional surface is constant. The problem involves a determination of the most efficient taper to secure an adequate total penetration in combination with a maximum frictional carrying capacity per unit of surface area.

ART. 56. DRIVING AND LOADING TEST PILES

Concrete piles have been in use so short a time comparatively that no standard practice has yet been developed with reference to the allowed settlement of test piles for a given loading. It may be desirable, therefore, to state a few examples of such specifications. In one case where the piles were to be driven

through materials ranging from quicksand to stiff clay, two test piles were required for each pier, the settlement in seven days under a load of 60 tons per pile being limited to ¼ inch. In another case test piles 35 feet long were not to settle over ½ inch in 24 hours under a load of 40 tons. The building code of a certain city states that the allowable load on concrete piles shall be taken as one-half of the load which shows no settlement for 24 hours, and a total settlement not to exceed 0.01 inch per ton of test load. Still another specification requires that not more than ¼-inch settlement shall occur on any one of six test piles for a building foundation under a load of 40 tons each. The piles varied from 30 to 40 feet in length and penetrated sandy soil underlaid by irregular strata of soft blue clay alternating with strata of stiff material. A load of 25 tons was assumed for the design.

In 1913 the city of Chicago required that for cast-in-place piles, test loads shall be applied on at least two piles in different locations and as directed by the Commissioner of Buildings, not less than three piles being driven at each location. The pile to be loaded is to be placed first; within 6 hours a second pile, and within 20 to 24 hours a third pile are to be placed at distances from the first not to exceed twice the greatest diameter of the pile, the measurements being made between centers. The tests are not to be made until 10 days after the placing of those which are to be loaded. The remainder of the test is to be the same as for pre-molded piles. In order to be certain that the kind of cast-in-place pile is adapted to the local subterranean conditions, it is necessary to excavate one or more piles. In some cases it may be necessary to drive steel sheet-piling around it to exclude the ground water in order to make the excavation.

Another city adopted specifications in 1913 for the test piles of a bridge foundation, requiring a balanced platform to be built on top of each test pile, and to have level readings taken on a rod set on a steel dowel grouted into the pile. For each test nine readings are required: Before the platform is placed; immediately after a 30-ton load is placed; 36 hours after this

load is placed; after the load is increased to 40, 50 and 60 tons respectively; 36 hours after the load is increased to 60 tons; after the load is reduced to 30 tons; and immediately after the entire load and platform have been removed. To be acceptable, the pile is not to show a settlement exceeding 1/4 inch between the first and third readings, exceeding 3/8 inch between the first and seventh readings or exceeding 5/16 inch between the first and ninth readings. The safe load is to be taken as one-half of the load which causes a settlement of 3/8 inch, and if this load is less than that originally assumed for the design, additional piles are to be driven so as to make the combined capacity of a group of piles adequate for the imposed load.

The following is the record of a loading test for a concrete pile in pier 19 of the reinforced-concrete viaduct on the Pittsburgh and Lake Erie Railroad referred to in Art. 49. The pile was 26.2 feet long below cut-off, the length of pre-molded pile used being 23 feet. In driving the casing the average penetration under the last five blows of a 3000-pound hammer, falling 15 feet was 0.45 inch. The loading was begun at 7 A. M. on Sept. 6, 1912. The loads in tons and corresponding settlements in feet are as follows: 18.5, 0; 27.0, 0.003; 32.0, 0.004; 35.0, 0.006; 38.5, 0.006; 45.0, 0.006; 52.0, 0.008; 57.0, 0.008; 59.0, 0.008; 60, 0.013 (Sept. 7, 2 P. M.); 60.325, 0.013; 60.325, 0.013 (Sept. 9, 8 A. M.). After removing the load, two-thirds of the settlement was recovered, leaving a permanent set of only 0.004 foot, or 0.05 inch. A test well 8 feet distant indicated that the pile penetrated 10 feet of cinder fill, 5 feet of dark mud, 3 feet of sand, 4 feet of gravel, 3 feet more of sand, while its foot rested on another stratum of gravel which is 4 feet deep.

The loading tests of two pre-molded piles driven by very heavy drop-hammers have been published. One was driven by a 7000-pound hammer to a depth of 27 feet 2 inches through silt, sand and gravel. A test load of 63 tons caused a settlement of only 1/8 inch at the end of two weeks. Another pile driven by a 12,000-pound hammer to a total penetration of 30 feet, upon being loaded with a weight of 72 tons showed no settlement at

the end of six months. The use of such heavy hammers was referred to in Art. 50.

While experience has shown that in most conditions of the ground the phenomena of pile driving give a fair measure of the bearing power, there are others to which this statement does not apply. Some moist clays are practically incompressible but, being plastic, the piles displace the material and force the surface upward elsewhere. This movement may be so small as to escape observation unless levels are carefully taken. In such a case the loading of test piles will reveal the true conditions. For example, a pile required 30 blows of a steam-hammer, having a striking weight of 3000 pounds and a stroke of 30 inches, to produce the last inch of penetration while the total penetration was only 9 feet. The ground was "ordinary yellow clay which was moist but not wet, and fairly solid." Under a load of 20 tons the settlement was $3\frac{1}{8}$ inches; for 25 tons, 5 inches, increasing to $5\frac{3}{8}$ inches the next morning; and for 35 tons, 7 inches, which increased to $7\frac{13}{16}$ inches the following morning. In subsequently testing a group of four piles it was observed that some of the adjacent unloaded piles also sank during the progress of loading, but rose after the maximum load had been in position for a time. Level readings taken over the whole area of the excavation revealed the fact that the volume of clay forced upward was practically equal to the volume of the piles beneath the surface. These tests led to a change in the type of foundation adopted.

ART. 57. SPECIFICATIONS

In Art. 40 extracts are given from GREINER'S Specifications which relate to timber piles; the following paragraphs are taken from the same source and relate to concrete piles.

88. Concrete piles, when reinforced and designed so that they may be handled and driven with steam-hammers in the same manner as timber piles, and when of the specified quality and sizes driven to refusal, may be subjected to a maximum load not in excess of 24 tons when used for railway bridges, all movable spans, arches and high abutments, and 30 tons when used for other foundations. Concrete piles molded in place without

metal reinforcement should not be used in water or ground so soft as not to give firm lateral support. When they are molded in a strong metal shell, previously driven to refusal and which remains in place after concrete has set, the safe loads when piles are completely embedded in firm earth may be taken the same as specified above for reinforced piles. When their design is such or the conditions are such as to necessitate the piles being jetted down instead of driven, the safe load should be not more than specified above or more than one-quarter of the failure load as determined by actual tests. When concrete piles act as columns they shall be designed as columns.

135. Concrete piles shall be of portland cement concrete in the proportion of 1 cement, 2 sand and 4 broken stone, varying in size from ¼ to 1 inch. They shall be constructed strictly in accordance with the plans but when their construction is not shown thereon they shall be of a type suitable for the conditions, and which will meet with the approval of the engineer. Their minimum diameter at tip and maximum diameter at butt shall be same as specified in paragraph 133 for timber piles. When driven through hard ground they shall be shod with steel points of approved design. When subjected to the maximum loads specified in paragraph 88, they shall go to rock or shall have an average penetration under each of the last 20 blows of a steam-hammer not in excess of that determined from the formula $S = HW/45{,}000 - 0.1$. In case this maximum penetration cannot be obtained without injury to the piles, or on account of the impracticable length required, the number of piles shall be increased until the load on each shall not exceed the amount indicated in the following formula for piles supporting railway bridges, all arches and movable spans.

$$P = \frac{1.06WH}{s + 0.1}.$$

For other structures the above load may be increased 25 percent. When concrete piles are jetted in place they shall either go to rock or to a solid stratum, in which case they shall be tested with steam-hammers and the set and loads shall not be greater than above specified. When piles are placed by other means than by hammers and jetting and when they are of such design as not to permit of them being driven same as timber piles, the safe loads and numbers required shall be determined by tests to failure as directed by the engineer, the expense of the tests to be borne by the contractor and included in his cost.

Specifications for constructing and driving pre-molded concrete piles may be found in the Manual (1921) of the American Railway Engineering Association.

CHAPTER V

METAL AND SHEET PILES

Art. 58. Tubular Piles

Some of the problems relating to underpinning and the foundations of buildings in New York City led to the introduction in 1901 of a patented pile, which consists of a steel pipe filled with either plain or reinforced concrete. The steel pipe or casing can be of any diameter of which pipe and well casings are manufactured, but the most usual sizes are 12 inches inside diameter and 9 inches outside diameter. The thickness varies from ¼ to ⅜ inch. Since, in underpinning, the head room is generally limited, the casings are designed to be driven in sections from 5 to 20 feet in length. The ends of the sections are machined so as to be truly perpendicular to the axis, thus securing a true alignment of the pile, and a uniform bearing of the metal. Inside sleeves are provided to hold the sections together and they have a driving fit in the pipe. Means are provided to prevent the sleeves from moving under the blows of the hammer while driving the pipe, and their length is not less than twice the inside diameter of the casing. The lowest section bears on the shoulder of a hollow conical shoe of cast iron or steel which is fitted with a hole for a water-jet, if required. The casing is driven like sheet piles with a steam- or pneumatic hammer, usually without leads, the head of the casing being protected by a cap. In some ground, especially in sand, the casing is driven without a shoe, and the sand is removed through the pipe as the driving proceeds. It is claimed that such a pile has been driven to a depth of 80 feet with perfect alignment.

When the casing has been driven, a hollow steel tube of considerable strength is thus provided which is then filled with

concrete. When the concrete is to be reinforced, sleeves connected to the casing are provided which hold each reinforcing rod in place without any lateral play. The pile is built up as it is driven down and if any material length projects above the ground it is cut off and used on another pile. Before the pile is filled with concrete an electric light can be lowered to ascertain if the true alignment of the casing has been maintained. In underpinning, the casing is often forced down by means of a hydraulic or a screw jack. If driven into soft soil without a shoe, the concrete may be rammed so as to form a bulbous foot to increase the bearing area.

Experience indicates that if the earth surrounding the piles remains undisturbed, the casings may last for many years. These pile casings are not good, however, when exposed to the action of moving water or air, owing to rapid rusting. In designing piles, careful consideration should be given also to the probability of injury due to electrolysis and methods of protection against it. In the trade the casing described above is known as the Simmons sectional concrete pile casing. Piles of this kind have been used up to 85 feet in length.

Figure 58*a* shows a wall pier of a 12-story office and loft building built in 1912–13 in New York City in which three tubular piles support a wall column seated on an I-beam grillage. The inside diameter of the steel tubes is 12 inches, and they are spaced 2 feet between centers. Additional piles are driven between the clusters to carry the walls between columns. Most of the interior foundations have clusters of four piles spaced 2 feet apart. All the tubes are made in two sections, connected by a cast-steel inside sleeve tapered slightly at the ends to make a driving fit and provided with an exterior horizontal rib in the middle against which the pipes take bearing. The rate of driving with a steel hammer varied from 40 to 200 feet in one eight-hour shift. They penetrated through some sand, about 8 feet of mud, 5 feet of hard clay, 25 to 35 feet of fine wet sand and gravel, to the irregular surface of the rock, which in most places was overlaid by about 2 feet of hardpan. The interior of the pipes was cleaned out every 5 to 20 feet by the use

of air pressure at 150 pounds per square inch delivered through a 2½-inch pipe without a nozzle, and which blew out the sand, lumps of clay and ground water high into the air. From one to five 2-inch reinforcing rods were set with a clearance of about 1 inch from the pipe and driven to a solid bearing. Their tops were also arranged to bear firmly against the cast-iron cap.

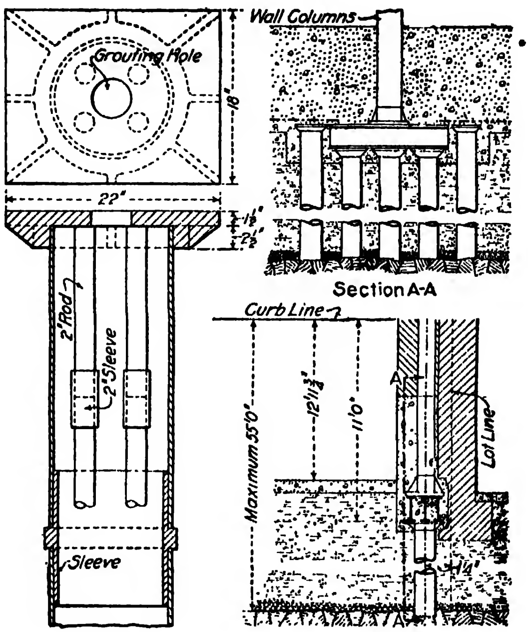


FIG. 58a.—Tubular Piles.

The concrete filling is a 1-2-4 mixture, and the piles were proportioned for loads of from 56 to 80 tons each. The stresses allowed for the steel and concrete are 4200 and 350 pounds per square inch, respectively. The section areas of steel vary from 17.8 to 30.2 square inches and of concrete from 109.9 to 97.5 square inches. A light framework containing templates at the top and bottom, and thoroughly braced was used to secure accurate location and alignment. It will be observed that the

details of these piles differ from those described in the preceding paragraphs.

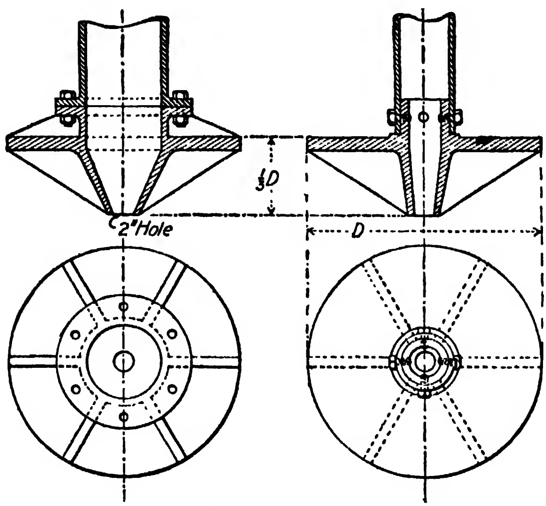
If the cleaning out of tubes or casings during the process of sinking causes trouble in the settlement of adjacent buildings, they may be driven to a firm bearing on the rock or hardpan and cleaned afterward by the dry-blow-out process. A bag of dry cement may then be placed in the bottom and the reinforcing rods placed in position. The casing is filled with water to resist any external pressure, if necessary, and after the cement has set, the casing may be pumped out and filled with concrete. Sometimes the jet which aids in sinking the tube and scouring out the interior is immediately afterward connected to a tank of grout under air pressure, and by discharging it at the bottom the grout displaces the water and sediment and makes it overflow. If a pile does not extend to rock, but is jacked down to a sufficient penetration in sand or gravel, the jack is applied again after the concrete has set, in order to force it to the required resistance. Under such conditions, a pile may sink from 3 to 6 inches further. In this manner there is more certainty of distributing a given load equally among several piles.

In one instance borings showed that a bed of quicksand 25 feet deep overlaid a stratum of very coarse gravel charged with water under a high head. After an 8-inch tube was driven down until it rested on the gravel and was cleaned out with a jet, a 1-inch pipe perforated at the bottom for 2 feet was driven 3 feet into the gravel. Grout was forced through the pipe to form a solid footing of grouted gravel and to seal the tube, which was then pumped out and filled with concrete. A test load of 35 tons caused no appreciable settlement.

The diameter of tubular piles, for use in underpinning and for some other suitable conditions has been increased considerably over those stated in this article. When the diameter is large enough to admit a workman to excavate the interior by hand, they are generally sunk by the pneumatic process. The larger sizes are preferably regarded as pneumatic caissons rather than pneumatic piles (for further details see Chap. XVI).

ART. 59. DISK AND SCREW PILES

A disk pile is one which has a disk attached to its foot to provide a larger bearing area. Disk piles have been used principally in ocean piers and wharves, where the total penetration is not large and is subject to more or less variation. The minimum penetration should not be less than about 6 feet below any possible scour. The disk is a casting which consists of a horizontal circular plate, stiffened by a number of radial ribs and connected to a central hollow stem, as shown



FIGS. 59*a* and *b*.—Two Forms of Foot of Disk Pile.

in Figs. 59*a* and *b*. The former illustrates the connection of the disk to a flanged cast-iron pipe which forms the body of the pile, and the latter the connection to a steel pipe. The upper part of the stem is cylindrical, while the lower part is conical so as to form the nozzle of a water-jet or to permit a water-jet pipe to pass through it. Sometimes the ribs on the upper side of the disk are made higher than the lower ones, their edges being inclined at an angle of 45 degrees. The disk pile can be used only in sand or soft material which permits sinking by the water-jet. If some material is encountered

which is not easily displaced by the jet alone the pile may be rotated to cause the ribs to act as cutters. In THEODORE COOPER'S General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges is given a table of the minimum sizes of pipe for corresponding diameters of disks. The diameters of disks range from 1.75 to 4 feet, those of the cast-iron pipe from 8 to 14 inches, with a thickness

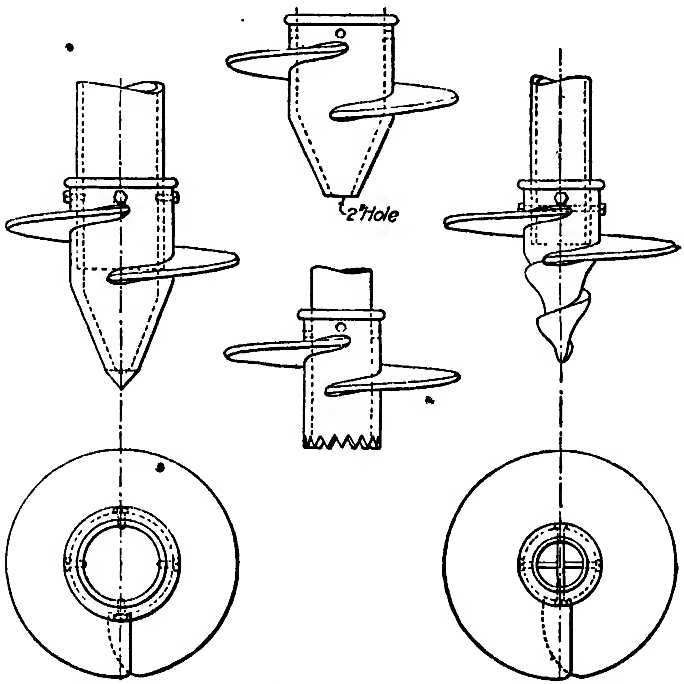


FIG. 59c. FIGS. 59d and e. FIG. 59f.
Four Forms of the Foot of a Screw Pile.

of 5/8 to 1 inch, and of the steel pipe from 6 to 10 inches, the thickness being 1/2 inch in all cases. The thickness of the disk plate, ribs and thickest part of the stem are not to be above 1 1/4 inches for a diameter of 2 feet or less, and 1 1/2 inches for larger diameters of disk. The ends of the cast-iron pipe sections are to be machined so as to secure perfect alignment.

A screw pile is one which has a broad-bladed screw attached to its foot to provide a larger bearing area. The use of the

screw pile is similar to that of the disk pile. The form of the screw casting is illustrated in Figs. 59*c* to *f*. The pitch of the screw varies from one-third to one-sixth of its diameter, the pitch adopted in any case depending upon the difficulty of securing penetration. The points of the screws are also varied, the gimlet point being suitable for gravel, the blunt point for sand, the hollow conical point for the use of a water-jet in sand and gravel and the serrated point for soft rock or coral. The dimensions of the shaft of the pile, and of the screw and its connections, must be carefully designed to furnish the torsional strength required to sink the pile into position. In one case where the frictional resistance was so great as to break several piles by torsion, it was discovered that by discharging a water-jet on the upper surface of the screw blade the friction was reduced so that the sinking could be accomplished without difficulty. After using the jet only about one-tenth as much power was needed to rotate the piles.

Screw piles were first used in 1838, and disk piles in 1856. They are unsuitable for deep foundations where the overlying material is soft or liable to scour, since it is impossible to brace the piles below the surface. It is quite probable that in future reconstruction these types will be replaced by reinforced-concrete piles.

ART. 60. SAND PILES

As stated in Art. 2, short timber piles are sometimes used to compact the soil and thus increase its bearing power. The same result may be accomplished at less cost by withdrawing the pile as soon as it is driven and filling the hole with sand. Such piles are called sand piles. They can be placed without regard to the elevation of the ground water-level, but cannot be used if there is any danger of scour, or in regions subject to earthquakes. The use of sand columns confined in wooden boxes to lower great weights has proved that they will sustain loads while developing relatively small lateral pressures. In order to have the sand pack firmly, it should be moistened

when placed in the holes and tamped. In case there is a slight settlement, the sand will readjust itself and maintain its stability. The method of consolidating the ground by ramming its surface and mixing sand with it during the operation is far less effective, since a hard crust is thus produced which transmits the pressure only to a very short distance below the surface. This difference may be proved by applying test loads and noting the settlement under a time test extending over a month at least.

In building a warehouse in Salina, Kan., 7-foot sand piles were placed by drilling holes through the sandy loam with a post-hole augur. The sand piles were stopped 3 feet below the ground surface and the space above filled with concrete. Load tests made on top of the sand showed ¼-inch settlement under loads of 8000 to 11,000 pounds and 1-inch settlement under loads of 11,200 to 14,400 pounds.

The "compressol system" is somewhat analogous to sand piles in first forming a hole and then filling it with a different material. The hole is made by a heavy conical perforator having a sharp point which is successively raised and dropped until the hole reaches a hard stratum. If the compressed earth does not keep the water out, the hole may be lined with clay dumped in after each fall of the perforator. Boulders are dropped into the hole and rammed with a tamping rammer which is shaped like a cartridge, thus forming a layer at the enlarged bottom of the hole. Concrete is then deposited in batches and tamped in the same way. In this way a sort of rude concrete pillar is formed. The system was originated in France and is seldom used in this country. It is more economical to use concrete in the form of concrete piles as described in the previous chapter.

ART. 61. TIMBER SHEET-PILING

Sheet-piling consists of special shapes of piles driven in close contact to form a reasonably tight wall, in order to prevent the leakage of water and soft materials, or to resist the lateral

pressure of the adjacent ground. Sheet piles are made of timber, of steel and of reinforced concrete. Sheet-piling is to be distinguished from "sheeting" which is set in place or driven as the excavation proceeds, as in trenches or open wells. Sheet-piling is driven in advance of and usually beyond the final depth of the excavation.

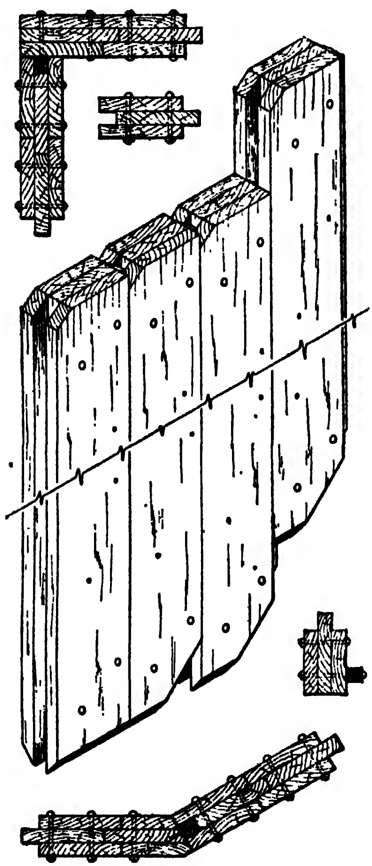
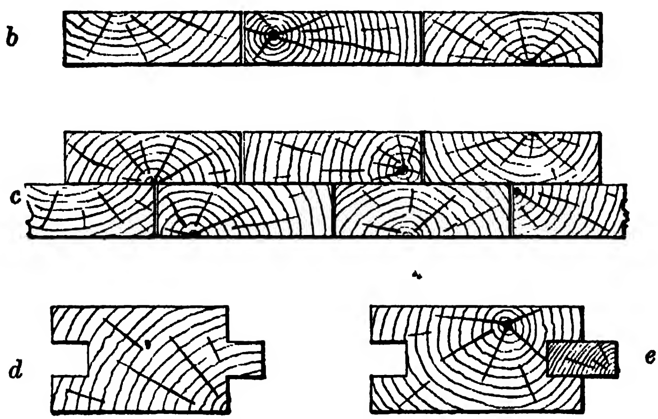


FIG. 61a.—Wakefield or Triple-Lap Timber Sheet-Piling.

The best form of timber sheet-piling is known as Wakefield sheet-piling and has been very extensively employed in this country. The patents secured in 1887 and 1891 have expired. It consists of three planks fastened together so as to form a tongue on one edge and a groove on the other (see Fig. 61*a*). The planks are connected by two bolts at intervals of about 6 feet, while spikes are used at intermediate points about 18 inches apart. It has been customary to use ½-inch bolts for planks from 1½ to 2½ inches thick, and ⅝-inch bolts for planks 3 to 4 inches thick. For sheet piles made of 1-inch boards, ⅜-inch bolts may be used. For the thin boards or planks, the tongue is made 1½ inches longer than its thickness, while for the thickest planks the length is the same as the thickness. The usual width of the planks is 12 inches, except for those less than 2 inches thick. By sizing the middle planks to a uniform thickness, a good fit can be secured between the tongues and grooves.

Experience has shown that these triple-lap piles are stronger to resist driving than if made of a single stick, this being due in part to the fact that cross-grain, knots or other defects in the three planks are not likely to be located at the same part of the length; and that some defects become visible and lead to the rejection of a plank which might not be visible in a single stick of the same total thickness. Other advantages of this form of pile are the absence of waste in forming the tongue and groove, and less tendency to warp or bend before they are driven. Figure 61*a* also shows how corners may be turned at



FIGS. 61*b*–*e*.—Sections of Timber Sheet-Piling.

a right angle by bolting and spiking a tongue to the face of a pile or at any other angle by fastening a tongue to a beveled side. It also illustrates how the foot of each pile is beveled on both faces, in order to drive plumb, and on one edge so as to keep in close contact with the adjacent one. The tongue should always be kept in the lead, otherwise gravel or stone may become wedged in the groove and damage the succeeding pile. If it is desired to drive the sheet-piling each way from the corner, the first pile should be constructed with a tongue on both sides, and sharpened so as to drive plumb longitudinally as well as laterally with respect to the lines of piling. The last pile in the center is constructed of the proper width and

acts as a wedge to tighten the line if necessary. When the sheet piling is to be driven to rock bottom, the middle plank should be cut off square at the end, so that water will not readily pass underneath the piling along the rock surface.

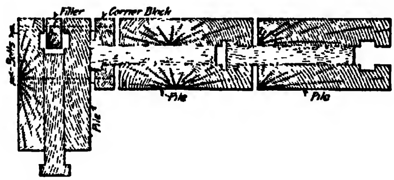


FIG. 61*f*.—Interlocking Wood Sheet-Piling.

Some other sections of sheet-piling are shown in Figs. 61*b*-*g*. In Fig. 61*b* a single row of ordinary planks are driven edge to edge. This arrangement cannot be used if it is necessary to secure a water-tight wall. In Fig. 61*c* two rows of planks are

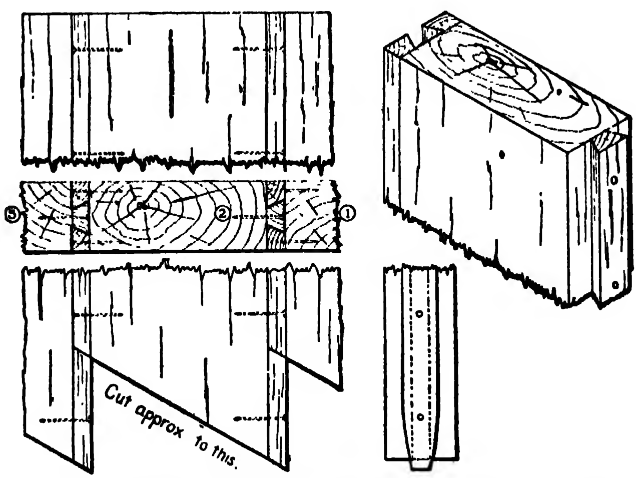


FIG. 61*g*.—Details of Timber Sheet-Piling with Dovetail Joints.

placed in contact and breaking joints. Figure 61*d* shows the cross-section of a sheet pile which is merely a plank with an ordinary planed tongue and groove. In Fig. 61*e* the plank has a groove cut on both edges and a tongue formed by nailing a strip or spline into one groove to form a tongue. In this

case, the tongue can be made of a different species of tough wood, like maple or elm, and carefully selected.

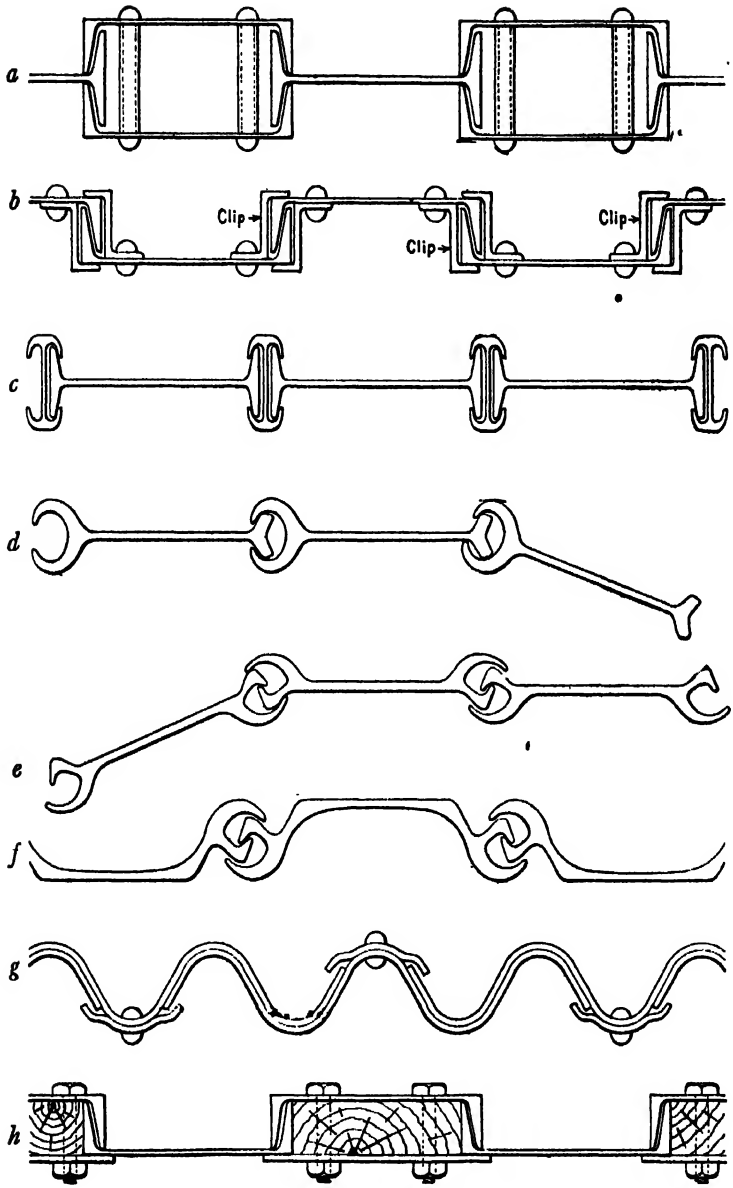
Figure 61*f* illustrates what is probably the tightest form of sheet piling. This type has been extensively used in levee work in Louisiana.

Figure 61*g* gives the details of construction for timber sheet-piling 4 inches thick according to the standard adopted by the Southern Pacific Co. The strips nailed to the planks are beveled to form a dovetailed tongue-and-groove joint. Sheet-piling built up in a similar manner is sometimes made as thick as 12 inches, and in exceptional cases 15 inches.

ART. 62. STEEL SHEET-PILING

On account of the difficulties encountered frequently in driving timber sheet-piling in hard ground without injury, and the large amount of bracing required to resist earth and water pressure during excavation and construction, an effort was made naturally to devise a sheet-piling of greater strength and stiffness, without excessive resistance to penetration. The need of this was emphasized by the increasing size and depth of foundation constructions, and the difficulty of securing water-tightness in quicksand. Rolled-steel sheet-piling was introduced to meet these conditions. The use of standard structural shapes in building up sheet piles gave such excellent results as to demonstrate the commercial success of steel sheet-piling. The first form of this class was employed in 1901, in the foundations of the Randolph Street bridge in Chicago and is similar to that shown in Fig. 62*a*. Alternate piles consist of standard I-beams, and the others are built up of two standard channels bolted together with pipe separators. The design is based on a foreign patent.

The next step forward was taken during the following year by the introduction of the Friestedt interlocking channel-bar piling. Each alternate pile consists of a standard channel, while each of the others is built up by riveting two Z-bars to a channel. An improved form known as the symmetrical inter-



FIGS. 62a–h.—Sections of Steel Sheet-Piling.

lock channel-bar piling is also manufactured in which a continuous Z-bar is riveted near one flange of every channel, and a short Z-bar clip is riveted near the other flange at the upper end only. This arrangement makes every pile alike and preserves its symmetrical head to receive the blows of the hammer (see Fig. 62*b*). The experimental work begun by LUTHER P. FRIESTEDT in 1899, which led to his patented form, and his efforts to extend its use by others, fairly entitle him to be known as pioneer of the steel sheet-piling industry in this country.

Another form which employs a standard structural shape was placed on the market in 1908. The piling consists of I-beams which are locked together by what is known as a locking bar. This bar consists of a small I-beam which, by extra passes through the mill, has its flanges bent into hook shapes as shown in Fig. 62*c*. One locking bar is attached to each beam at the mills by steel wedges and then the beam and locking bar are driven as one sheet pile. The beams differ slightly from the ordinary standard in having the outer corners of the flanges rounded.

Another class of steel sheet piles includes those which consist of a single shape formed by special rolls. Figure 62*d* shows a section of what is known as United States sheet-piling. At one edge of the web is a flange with a bulbous section and at the other edge is an open or slotted cylindrical flange. This type was invented some years before the Jackson and Friestedt types but did not come into commercial use until a few years after their introduction. The smaller flange of one pile enters easily the larger curved flange of the next one, the slot being wide enough to allow a considerable change in the direction of the web.

The Lackawanna sheet-piling introduced in 1908 is also a special rolled section as illustrated in Fig. 62*e*. Both flanges are alike, the section being symmetrical with respect to a central transverse plane. The diagram shows how the flanges of adjacent piles engage each other to form a double interlock. The shorter flanges may be said to engage like hooks, while the larger ones act like guards. Where flexural strength is of

primary importance, the web is curved as indicated in Fig. 62*f*. It will be noticed that the web is bent until a considerable portion of its width touches the plane which is tangent to the curved flanges of the adjacent piles. A large bearing surface against supporting timber wales is thus obtained.

Corrugated-steel sheet-piling, which was first used in 1907, illustrated in Fig. 62*g*, permits the use of very thin plates. A double thickness of metal is provided at each joint. The width of sections can be varied to suit the conditions of driving. Figure 62*h* shows sections of the Gould sheet pile in which alternate piles are standard channels, as in the Friestedt type, while each of the others consist of a channel and a plate bolted together with a spacing timber between, the timber being of such a size as to provide for the interlock at each edge. A U-shaped bent-plate shoe is bolted to the bottom of each combination pile to protect its foot during driving. There are several other patented forms which have been used to a limited extent.

The general method of forming a corner pile is by cutting an ordinary sheet pile longitudinally into two halves, and then riveting them to a structural steel angle. In some cases a sheet pile has its web bent to a curve of short radius to form a corner pile. At the junction between a longitudinal wall and a transverse wall on one side of it, a half section like that used at a corner is riveted to the web of a whole section by means of two angles.

It will be observed that each type of sheet-piling has interlocking edges to prevent a pile which is being driven from pulling apart from the one driven previously. This is an important advantage not possessed by timber sheet-piling. The tensile strength of the interlock enables steel sheet-piling to resist a considerable lateral pressure without the aid of transverse bracing. In the construction of some very large cofferdams it has been possible to take advantage of this feature by constructing a row of pockets to be filled with excavated material so as to avoid the use of transverse bracing in the large interior area (see Art. 73). The direct tensile strength of

interlock for five types of steel sheet piles for sections weighing approximately 40 pounds per square foot, with one exception, were found by tests in 1908 to be 9746, 7769, 3842, 3362 and 1094 pounds per linear inch. More extended tests later gave values of 9584, 7271, 6060, 3569, 2252, 2022 and 1502 pounds per square inch, the lowest three values relating to fabricated sections.

Not only do the different types of steel-piling vary materially in the tensile strength of the interlock, but also in the section modulus which measures the resistance to flexure between the horizontal wales or the frames which support them laterally, as well as in their least radii of gyration which indicate the resistance as a column while being driven. Each type is usually manufactured in different sizes and weights, for example, the United States piling has two sections with an effective width of 13 inches with weights of 38 and 42.5 pounds per linear foot, and a small section 9 inches wide weighing 16 pounds per linear foot. All the types permit some degree of flexibility in the interlock, being small in those using structural shapes and large in the special rolled sections, the maximum change of direction toward either side being about 20 degrees. This arrangement permits piling to be driven in a curved line, or to avoid boulders encountered in a proposed line.

Steel sheet-piling is employed in cofferdam construction, for retaining walls, to protect adjacent buildings during excavation, to line shafts in quicksand, to line open wells for building piers, as well as for dams and other hydraulic constructions. It is practically used for the same purpose as timber sheet-piling, but the results secured by it are much more certain. Only one wall is often required to secure water-tightness which would require two walls of timber sheet-piling under the same conditions. Single lengths of steel sheet piles 52 feet long have been successfully driven with sections having a weight of about 40 pounds per linear foot. Spliced lengths of 75 feet were employed in the cofferdam around the wreck of the U. S. Battleship *Maine* in Havana Harbor. It is not necessary to form a splice by bolting or riveting but merely to abut the

pieces and break joints with the adjacent piles. Spliced lengths up to 96 feet have been employed in exceptional cases.

Further details regarding the properties and uses of steel sheet-piling, proposed specifications and an account of its historical development may be found in a paper by L. R. GIFFORD on Steel Sheeting and Sheet-Piling, and its elaborate discussion, in the Transactions of the American Society of Civil Engineers, vol. 64, pages 441–525, September, 1909.

ART. 63. CONCRETE SHEET-PILING

In the construction of piers or wharves where sheet-piling forms a part of the permanent structure, reinforced-concrete sheet piles are employed. Sometimes they are rectangular in cross-section and are driven in as close contact as possible, the foot being beveled on one edge like timber sheet piles. The larger sizes have tongues and grooves on the edges, the sides of the grooves being splayed so as to engage the tongues more easily. Experience shows that it is not advisable to have a thickness less than 8 inches when a tongue and groove are used, as it is otherwise difficult to obtain the requisite strength for these details. Another plan consists in forming a semi-circular groove on both edges, thus forming a cylindrical space with the adjacent pile, to be occupied by the water-jet pipe during sinking and to be filled afterward with grout.

At the terminal piers at Brunswick, Ga., reinforced-concrete sheet piles 18 inches square were used for the bulkhead at the basin. They were 45 feet long, beveled at the foot to 12 by 18 inches and weighed 7 tons each. Four ¾-inch square reinforcing bars extended the full length, and for the lower two-thirds of the length, two 1¼-inch bars were added in trussed form to help in resisting the maximum bending moment. In 1910 at the Norfolk Navy Yard the sheet piles were 18 by 24 inches in section with tongue and groove, and 55 feet long. The most extensive use of concrete sheet-piling up to that date occurred in the Galveston causeway, begun in 1909, 9808 piles being required. They were 10 by 18 inches in section, grooved

on both edges and grouted together after being driven. To improve the protection of the reinforcing rods, it is desirable to place them farther from the surface on the water side than on the other side of a pile.

In order to combine the tensile strength of the interlock for steel sheet-piling with the freedom from corrosion of concrete piles, sections have been designed in which a steel pile is cut longitudinally through the web and these halves are cast into a concrete pile on opposite edges. After the piles are driven, the grooves containing the steel interlock are cleaned out with a jet and filled with grout. Another arrangement consists in making a combination pile by enclosing an entire steel sheet pile within a concrete pile, the joints being grouted in a similar manner after driving the piles.

ART. 64. DRIVING AND PULLING SHEET-PILING

The construction of a single wall of timber sheet-piling is illustrated in Fig. 64*a*. Each row of vertical guide piles supports several horizontal timbers called "wales," against which the sheet-piling is driven. An outside wale is usually bolted to the upper inside wale in order to hold the sheet-piling in line. It will be noticed that a pair of short leads is attached in front of the ordinary fixed leads of the pile-driver in order to bring the hammer directly over the line of sheet-piling. The light-weight steam-hammers especially designed for driving sheet-piling give the best service, most of which are operated without any leads, being held in position by the boom of a derrick. Numerous illustrations and descriptions may be found in the catalogues of manufacturers. They will drive piles to a greater depth, and without brooming, splitting or other injury. To secure a good job, the piling must be very carefully driven. If a pile is injured by some obstruction in driving, it is generally better to replace it at once than to attempt some other means of repairing the wall to make it water-tight. In case sheet-piling has to be driven through a shallow deposit of silt or sand to a rock bottom and it is desired to secure a close fit where the

bottom is not level, a sheet pile may be sharpened to a knife edge, driven until the edge is broomed to contact throughout, then pulled up, the end cut to the proper form and finally redriven. After all the sheet-piling is in place, the hammer should be placed on each pile in succession to secure closer contact with the rock by slight brooming at the foot.

Timber sheet-piling is sometimes used to form cofferdams for piers, and is left in place as a protection from scour around the heads of the bearing piles which support the piers. This was done on some of the piers of the Vancouver bridge of the Portland and Seattle Railway. Wakefield sheet piles were built up to a maximum length of 68 feet. In order to secure the necessary penetration through the sand, which varied from 45 to 59 feet below the cut-off, the sheet piles were sunk with the aid of a water-jet. It would have been impossible to drive them without the jet aiding the steam-hammer.

In driving sheet-piling through hardpan that could hardly be penetrated with a pick, a water-jet under 400-pound pressure was successfully used in Seattle.

An ingenious arrangement in which a guide or pilot tube is utilized both as a water-jet tube and as a guide for each pile is described in Engineering News, vol. 70, page 552, Sept. 18, 1913. The tube, which has a flattened oval section, engages the groove in the pile already driven and the adjacent groove of the pile to be driven. The tube is afterward withdrawn and the space filled with a hardwood spline.

Steel sheet piles are driven generally by steam-hammers, the weights of which are proportioned to that of the piles to be driven. The double-acting steam-hammer is very effective for this purpose because of its rapidity of action, which keeps the pile practically in constant motion, and because it can be handled for this purpose without leads. The lightest hammer of several designs can be handled by one man, and sometimes a step is attached to the hammer frame, so that the weight of the man who operates it may be added while driving. Long sheet piles with heavy sections are driven with heavy steam hammers and pile-drivers.

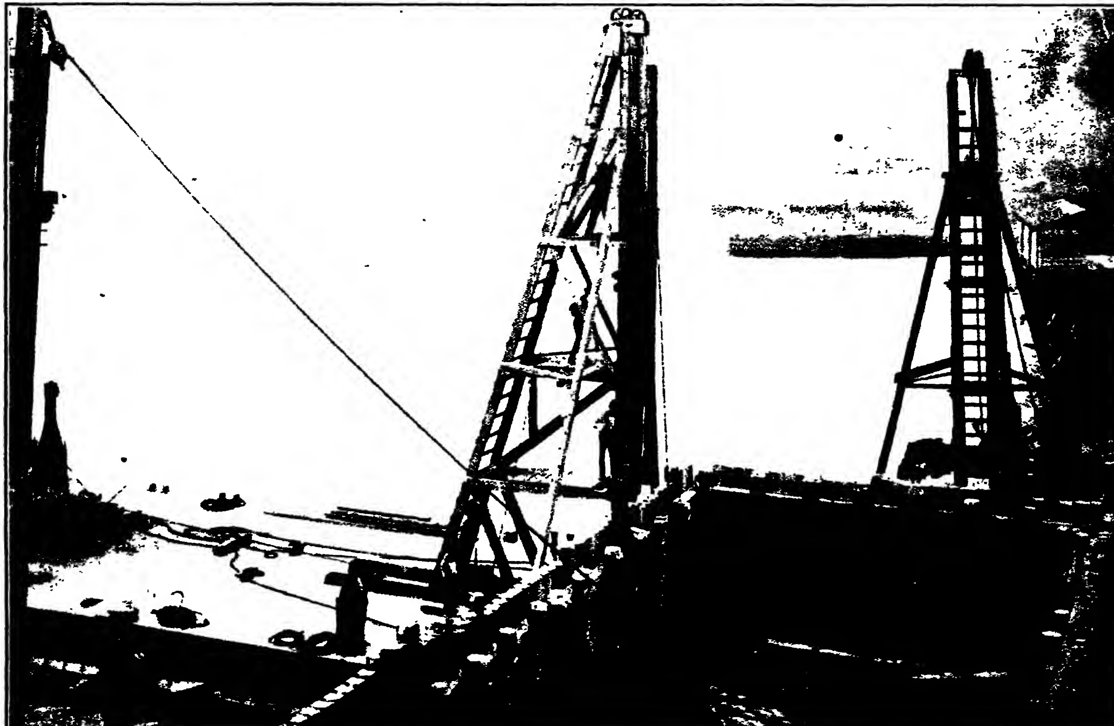


FIG. 64a.—Driving Wakefield Sheet Piling for Cofferdam of Pier No. 7 of Illinois Central Railroad Bridge at Gilbertsville, Ky., Sept. 23, 1904.

(Facing p. 202.)

The comparative resistance of different makes of steel sheet-piling is indicated by driving tests with five types of piling at Black Rock Harbor in 1908. The number of blows of the steam-hammer per square foot of piling was found to be 13.9, 14.4, 12.1, 19.6 and 22.9. All of the piling weighed about 40 pounds per square foot, except one type which was lighter.

To protect the head of a steel sheet pile, especially in hard ground, a cap is generally employed which contains a wooden cushion or driving block. Its base contains grooves or sockets which fit over the pile. The elaborately illustrated catalogues of manufacturers of steel sheet-piling show plans and sections of caps which are designed for each type of pile. There is usually a transverse as well as a longitudinal groove in the cap so as to fit a corner pile or a junction pile as well.

In driving through material with a large proportion of sand or clay, the interlocks seal themselves with the material penetrated. Occasionally, strips of wood are driven into the openings, which by swelling help to make the joint water-tight above the bottom of the water. Sawdust or wood pulp may also be used to stop leaks. The special rolled sections offer less resistance to driving on account of the absence of rivets or bolt heads; and hence they may also be pulled more easily. In sections like the United States piling, the bulbous flange should be kept in the lead. Under ordinary conditions steel sheet-piling may be pulled and redriven a number of times and finally has considerable scrap value, thus frequently making the cost less than for timber sheet-piling which can ordinarily be used only once. Experience has shown instances in which steel sheet piles driven into hard ground could not be used over again, and in exceptional cases, it has been impossible to pull it, making it necessary to dredge away some material alongside and to bend it down on the bottom to avoid interference with navigation. Whether this result was due in any measure to improper driving remains uncertain.

Under certain conditions, it is not desirable to drive each pile to its full penetration at one operation. In order to maintain good alignment, or to facilitate closure, it is often advanta-

geous to set up a considerable number of sheet piles and then drive them several feet at a time in succession, repeating the operation until the desired penetration is reached. The same method of driving ahead some distance may be used successfully in avoiding injury to piles when boulders are encountered, if they are not too large. The damage thus becomes local and limited in extent. The water-jet may also be used to aid in displacing boulders. Sheet piles should be carefully handled in transportation, for with a small clearance in the interlock, a bend or kink due to careless handling may cause so much friction that the pile refuses to move on reaching a hard stratum, and may result in crippling the pile, if driving is continued.

Steel sheet-piling has been successfully driven through submerged logs, old timber cribs, brick, stone and other débris in made ground. If considerable cribwork or logs have to be penetrated, it may be more economical to construct a special chisel attached to the end of a timber, and to cut the timbers with the aid of the chisel and pile hammer before inserting the sheet pile. The construction of such a tool is described in the Engineering Record, vol. 66, page 704, Dec. 21, 1912.

To drive sheet-piling below the leads of a pile-driver, a follower may be constructed for the purpose by riveting to the web of a piece of piling, of the proper length, two plates or channels which project below its web and engage that of the sheet pile to be driven.

Because steel sheet-piling is usually used a number of times and after becoming too badly bent for further use has considerable junk value special attention has been given the subject of pulling the piling. At the present time most piling is pulled by means of the inverted steam-hammer. Other methods used are block and tackle attached to a derrick boom, gallows frame or gin pole and the hydraulic jack.

In 1914 a special machine was developed in England for pulling piles and at about the same time in this country the ordinary steam-hammer inverted was first used in drawing the sheet-piling of the Harlem River section of the New York subway. The rigging consisted of a wire-rope sling suspended

from the crane hook and supporting the hammer. A heavy strap of steel passed over the anvil block of the hammer and was fastened to the pulling straps pinned to the pile.

Piles as long as 72 feet have been pulled with the inverted steam-hammer. The pulling resistance varies between wide limits, depending on the class of material through which the piling is driven and the tightness of the interlock. Where the piling has been overdriven and bent, it may be impossible to pull the same. Except in key piles the pull will usually not exceed 40 tons, while a 15-ton derrick equipped with a steam-hammer will develop the equivalent of a steady 75-ton pull.

In pulling the key pile of the New York cofferdam described in Art. 73 the necessary pull was 350 tons, which was obtained by the use of block and tackle attached to a gallows frame.

A combination of water- and air-jet has been successfully used to loosen the material around sheet piling. A mixture of kerosene and heavy oil poured down the interlock may facilitate starting key piles. If the piling has to remain in place a long time, the pulling may be facilitated by lubricating the joints with graphite or some other material which will prevent corrosion of the interlocking joints. If concrete is deposited next to steel sheet-piling, which is to be pulled subsequently, it is essential to prevent contact between the concrete and steel by using tar-paper, or preferably light wooden sheeting with tongue-and-groove joints.

Occasionally, it is necessary to cut off steel-piling to an exact level. Where only a few pieces have to be cut and where time is not an important element, hack saws may be used economically. For larger quantities to be cut in the least time, the oxyacetylene flame is the most advantageous in operation and cost. The electric arc has been employed in some instances, but its cost is very high and it is difficult to handle on account of the intense light produced.

ART. 65. DESIGN OF SHEET-PILING

When sheet-piling is driven a short distance into the bottom and is supported at the water surface by wales and struts,

as illustrated in Fig. 65*a*, each pile may properly be regarded as a simple beam with a span d. Taking the weight of a cubic foot of water as 62.4 pounds and expressing distances in feet, let a sheet pile be considered 1 foot wide. The pressure in pounds per square foot at the depth d is $62.4d$, the total pressure on the pile is $31.2d^2$, distributed as shown in Fig. 65*a*; and since the center of pressure is at $\frac{1}{3}d$ from the bottom, the horizontal reaction at the surface is $10.4d^2$. By the principles of mechanics, the bending moment at any distance x below the

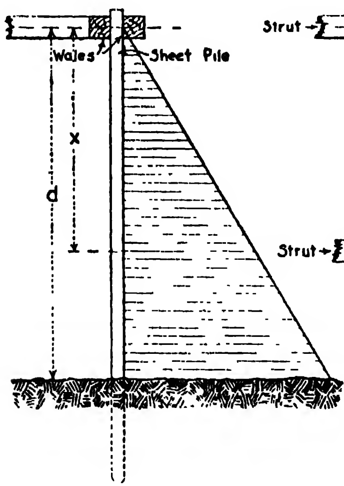


FIG. 65*a*.

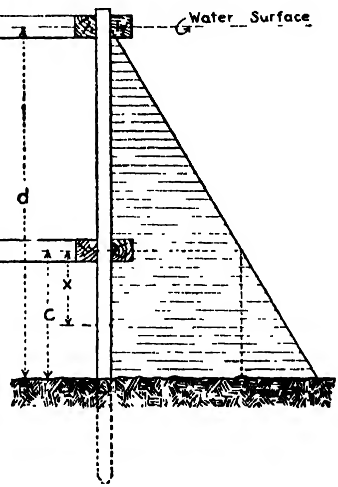


FIG. 65*b*.

surface is $M = 10.4d^2x - 62.4x \cdot \frac{1}{2}x \cdot \frac{1}{3}x = 10.4d^2x - 10.4x^3$. Placing the first differential coefficient dM/dx equal to zero, there is found $x = d/\sqrt{3}$ or $\frac{1}{3}d\sqrt{3} = 0.577d$, for the location of the maximum bending moment. The maximum bending moment is, accordingly, $4.00d^3$, expressed in pound-feet; or $48.00d^3$, expressed in pound-inches. If the total pressure be regarded as uniformly distributed over the pile, the value of the maximum bending moment is $3.90d^3$, or 2.5 percent less than the true value.

The strength of Wakefield sheet-piling must be regarded as that of three separate planks, since the longitudinal shear developed between them by flexure cannot be fully resisted by

the bolts or spikes which connect them. Such piles are analogous to deepened beams which also require better means than connecting bolts to develop their strength as a unit. The tests of columns composed of two or more sticks bolted together also show that in no case is the resistance materially greater than if each stick were acting freely (see JACOBY'S Structural Details, Arts. 43, 45, 49 and 50). Let it be required to find the thickness of Wakefield sheet-piling for a depth of water of 10 feet, the unit-stress in the outer fiber being taken at 1000 pounds per square inch. The resisting moment of the three planks 12 inches, wide is, accordingly, $1000 \times 12 \times 3t^2/6 = 6000t^2$ pound-inches, in which t is the thickness of each plank in inches. Equating this to the bending moment of $48.00 \times 10^3 = 48{,}000$ pound-inches, there is found $t = 2.83$, or $2\frac{7}{8}$ inches. In determining the commercial sizes required, account must be taken of the loss due to sawing as well as for planing the middle planks in the construction of sheet piles.

In Fig. 65*b*, the sheet pile is horizontally supported at the water surface and at an intermediate depth. Let $d = 16$ feet, and $c = 6$ feet. The pressure at a depth of 10 feet is 624 pounds per square foot, and at depth of 16 feet is $624 + 374.4 = 998.4$ pounds per square foot. Taking a width of pile of 1 foot, the pressures on its lower portion, represented respectively by the rectangle and triangle of the shaded area are 3744 and 1123.2 pounds. Treating the pile as a simple beam with a span of 6 feet, the reaction at the intermediate wale is $\frac{1}{2} \times 3744 + \frac{1}{3} \times 1123.2 = 2246.2$ pounds. The bending moment expressed in pound-feet at a distance x below this support is $M = 2246.4\ x - (312x^2 - 10.4x^3)$. In the same manner as before, the value of x which makes M a maximum is found to be 3.115 feet, while the value of the maximum bending moment is 3656.0 pound-feet. If the total load is regarded as uniformly distributed, the maximum bending moment is $4867.2 \times \frac{6}{8} = 3650.4$ pound-feet, which is only 0.15 percent less than the true value. Since the error decreases as the depth d increases, it is sufficiently precise for purposes of design to use the simpler approximate method of computation for all portions of a sheet

pile below the top span, for which the true value of the maximum bending moment is expressed by a simple term as given in the first paragraph of this article. If the values of d and c were 16.25 and 6.25 feet respectively, the approximate value of the maximum bending moment is 3999 pound-feet, which is practically the same as for the upper span of 10 feet.

If a timber sheet pile is built up as shown in Fig. 61*f*, the small pieces spiked to the main timber to form the tongue and groove should be omitted in computing the resisting moment of the section. The values of the section modulus for the commercial sizes of steel sheet piles may be obtained from the manufacturers. The corresponding width to be used in computing the bending moment per pile is the distance center to center of interlock when assembled.

The design of sheet-piling to resist earth pressure in which the material has more or less cohesion is not on a basis that is entirely satisfactory. The conditions vary so widely and often the material penetrated in any locality occurs in layers of different density or character that it is well to make the design so as to be on the safe side. Some engineers design all sheet-piling for hydrostatic pressure, increased by 50 percent or more for wet slippery material.

CHAPTER VI

COFFERDAMS

Art. 66. The Cofferdam Process

When, for some purpose, it is desired to exclude the water and expose a portion of the bottom of a river, lake or other body of water, a structure called a cofferdam is employed. This cofferdam is a temporary structure, practically water-tight and large enough to provide adequate room for working.

Defined, a cofferdam is a temporary structure used for excluding the water from a given site, or area, either wholly or to such a degree that, with a reasonable amount of pumping, the permanent substructure may be built within it in the open air, or that other work may be accomplished.

The building of the permanent substructure may include pile driving, placing grillages, building piers and abutments, etc., while other work may include the construction of dams, removal of sunken vessels, etc. Where the ground is saturated with water, cofferdams are sometimes used in placing foundations for buildings.

Cofferdams are usually built in place. They may be self-contained or may depend for strength on the natural bottom, as is the case where guide piles are used. Bracing may be used to resist the lateral pressure against the walls.

To obtain water-tightness the sides of the cofferdam must be tight and the soil on which the cofferdam rests must be impervious. If the latter condition does not exist, either the sides of the cofferdam must extend through the pervious material to an impervious stratum or else a layer of concrete must be spread over the bottom inside the cofferdam and allowed to harden before pumping is begun. Absolute water-tightness is seldom sought, it being cheaper to pump a moderate

amount of leakage than to go to the heavy expense of building a structure that will not leak. The cofferdam should be so designed that the combined cost of construction, maintenance and pumping shall be a minimum.

To depths of from 20 to 30 feet the cofferdam process will prove the best and cheapest method of founding bridge piers and abutments, but for depths greater than 30 feet, owing to the difficulty of properly bracing the cofferdam against the pressure of the water, as well as preventing heavy leakage, some other method is usually preferable. Cofferdams over 50 feet deep have been used in a few instances.

Cofferdams may be constructed of earth, timber, steel or concrete. They may be divided into five general classes: earth, sheet pile, crib, movable and miscellaneous cofferdams. These classes will be described separately in the following articles.

ART. 67. EARTH COFFERDAMS

Of the five classes the earth cofferdam is the oldest in origin and simplest in construction. Its use is usually limited to shallow water with low velocities of current. It is made of a bank of earth placed around the site to be enclosed, and of a thickness sufficient to furnish the required stability and to keep the leakage down to a small amount. The earth bank should be carried up 2 or 3 feet above the water-level with a width of at least 3 feet at the top, and with side slopes corresponding to the natural slope of the material. The embankment should preferably be composed of a mixture of clay and sand or gravel, but if clay is scarce the bank may be composed of sand with a clay wall in the center.

The amount of embankment may be somewhat reduced by using one or two rows of sheet-piling, in which case the cofferdam may resemble more or less closely the sheet-pile cofferdam described in later articles. As to whether in any given case the cofferdam should be classed as an earth or sheet-pile cofferdam will depend upon whether or not stability and water-tightness depend primarily upon the earth filling.

Where the depth of water is not more than 4 or 5 feet and the velocity of the current would wash away loose material, cofferdams may be made of ordinary canvas bags about half filled with a mixture of clay and sand. It is important that the bags shall be but partially filled, for otherwise they will not pack together closely.

A modern and up-to-date use of the earth cofferdam is found in the construction of the cofferdams of the West Neebish Channel of the St. Mary's River. In some places the depth of the water was far too great for the economical use of earth cofferdams and was justified here only by the extremely favorable conditions that obtained for placing the earth. Two subsidiary cofferdams were first constructed across the channel about midway between the main ones in order to stop the current and divert the flow to another course. "These temporary dams were about 1000 feet apart at the site of the channel and extended across the river from the mainland to the island, varying in direction to suit the contours of the river bed. They were built in 2 to 7 feet of water flowing 3 to 6 miles an hour. The construction of these dams stopped the flow of water in the West Neebish Channel of the river, so that the main cofferdams could be built in still water, and also laid bare a part of the site of the channel about 1000 feet long. In building these temporary dams, which varied from 4 to 10 feet in height, broken stone and rock were dumped from scows on the line of the dams until the force of the current was broken and the rock fill carried above the water. Sandy clay was then brought in and dumped on the upstream side of these rock embankments in order to silt up the openings and produce water-tight dams."

The main cofferdams which unwatered the 8600-foot section of the work were structures of unusual size. The upstream cofferdam was 1900 feet long and was built in water from 2 to 18 feet in depth. [1]"This cofferdam has a minimum width of 8 feet at the top which is 7 feet above the water, and has side slopes on the water side of about 1 on 1½, and of about 1 on

[1] Engineering Record, vol. 56, page 112, Aug. 3, 1907.

2 on the other side. The other main cofferdam is 8600 feet downstream from this one. It has a total length of 2600 feet, and in plan is arched slightly downstream against the water on that side of it. This cofferdam was built in water from nothing to 26 feet deep; it has a minimum width of 12 feet at the top, which is 6 feet above the water; its side toward the water is built on an average slope of 1 on 2, and the one on the other side of 1 on 2½.

"The construction of the upstream main cofferdam was started soon after the current of the river had been broken by the temporary dams. Sandy clay and mud excavated by the dredges at work on the adjacent sections of the channel were brought to the site in bottom-dump scows and deposited in place. When the banks thus formed had been carried up until the bottom-dump scows would operate no longer, the materials were loaded on flat-deck scows, and handled from these to place in the embankment by a clam-shell bucket on a derrick scow."

French engineers have made extensive use of the earth cofferdam for work on their various canals. In some work, on the Meuse Canal, described in Annales des Ponts et Chaussees, 1896, page 539, gravel ranging in size from about 1 to 4 inches—being the residue, after the sand was used, of material dredged from the canal bottom—was employed. Water-tightness was obtained by placing a layer of tan-bark over the water face. In some cases the head of water on the cofferdam was as much as 9 feet.

In place of earth, cofferdams are sometimes made of fascines. The cofferdam for a concrete dam at Milford, Conn., was made by forming brush into mats, which were sunk by loading with rocks, the layers of brush and stone alternating. To give water-tightness a layer of earth was placed over the upstream side.

Figure 67*a* illustrates the cross-section of the earth cofferdam with sheeting used in the construction of the Ohio River lock and dam 48, where the bottom was composed of sand. To break the current, a line of sheet-piling was first driven. Frames were then placed by a boat, as shown in Fig. 67*b*, and

connected to the sheet piling. The frame was built by moving the boat to the right until A cleared B, then plank X was raised to a vertical position and the rods placed. This operation was then repeated for plank Y. Before raising plank Z the lower

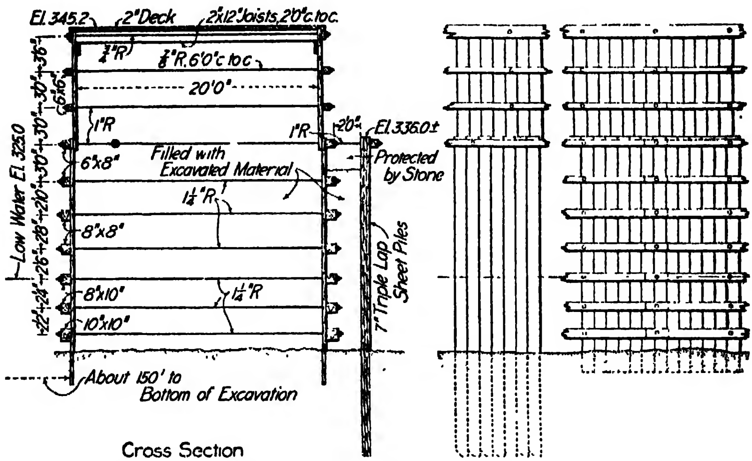


FIG. 67a.—Sheeting for Earth Cofferdam on the Ohio River.

wale BW was fastened, after which the section S was allowed to drop to the bottom and the lifting line transferred from S to S'. Vertical planking was placed against the frames and the

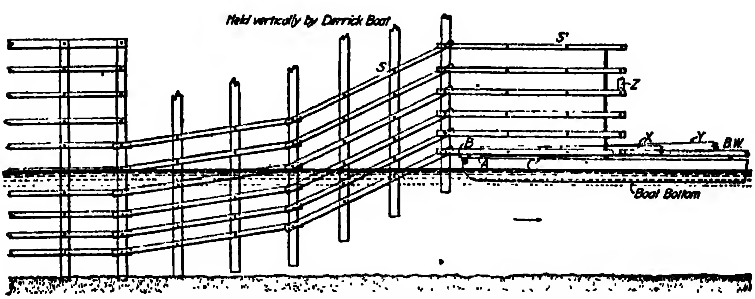


FIG. 67b.—Method of Constructing Ohio River Cofferdam

interior then filled with dredged material. Gravel was placed along the outside of the sheet-piling up to its top and on a slope of about 45 degrees; the space between the sheet-piling

and sheeting was also filled with earth, and finally sand was placed against the inside wall of sheeting up to the elevation of the sheet-piling top. This sand had a very gentle slope, running approximately 100 feet before reaching the elevation of the bottom of the sheeting.

In another cofferdam of this type (Fig. 67*c*) the method adopted consisted in laying horizontally and loosely the two separate sets of walings for a 20½-foot section on two corresponding frames or cradles lying opposite each other on the

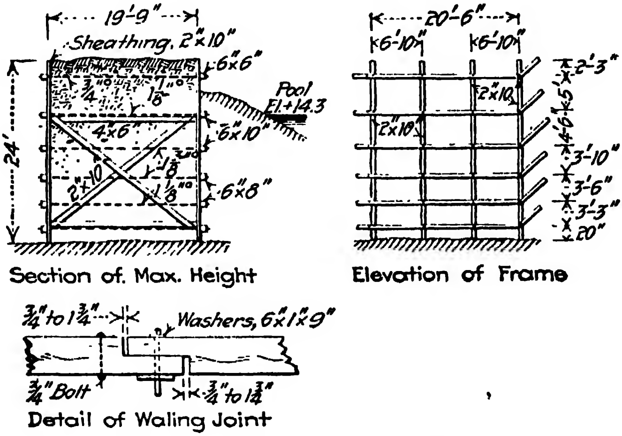


FIG. 67*c*.—Frame for Earth Cofferdam.

deck of the working barge. Each cradle was provided with projecting arms bolted on at distances corresponding to the spacing of the walings and supporting the latter when the cradles were set up. The two cradles were then lifted to a vertical position, the rods attached, as well as struts at top and bottom, and connection made to the preceding section. The frames were then lifted slightly allowing the cradles to clear and be lowered, after which the scow was moved so as to launch the framework.

ART. 68. WOODEN SHEET-PILE COFFERDAMS

The sheet-pile type may be considered as the standard form of cofferdam. It consists of rows of sheet-piling, usually not

more than two, extending around the site to be inclosed. The piling is held in place in various ways as described in the following articles. The sheet-piling serves the function of giving water-tightness to the structure, and to this end some form of intermeshing or interlocking piling is always employed. Strength to resist the pressure of the water outside is furnished by guide piles, frames or cribs, in addition to a large amount of internal bracing. The sheet-piling may be of wood or steel; at the present time (1925) the use of various forms of steel piling is rapidly increasing.

Where it is possible to drive piles some distance into the soil the sheet-piling is best supported by vertical guide piles and horizontal wales. The latter will not only furnish a guide for the sheet-piling while being driven, but will also add strength to the cofferdam, thus decreasing the amount of internal bracing necessary.

Double Wall with Guide Piles.—Figure 68*a* shows the details of this type of cofferdam. It is composed of vertical guide piles, horizontal waling and cap timbers, vertical sheet piles and a puddle filling. Rods are usually put in near the top to connect each pair of guide piles in order to prevent the filling from spreading the walls apart. If the top of the cofferdam is but slightly above water-level, struts are often placed near and parallel to the tie rods, serving to hold the two walls apart.

The bearing piles are driven more deeply into the earth than the sheet piles, the aim being to drive them far enough to develop the full transverse strength of the pile when acting as a free cantilever above the earth. The sheet-piling should be driven to a fairly impervious stratum to prevent leakage under the cofferdam. The space between the walls should be filled with earth, preferably an intimate mixture of sand and clay or gravel and clay, to form a puddle (Art. 77), which will materially assist the sheet-piling in making the cofferdam water-tight. This puddle should be placed in thin layers and thoroughly tamped in a damp state. Before placing the same it will usually be advisable to dredge out the soft material on

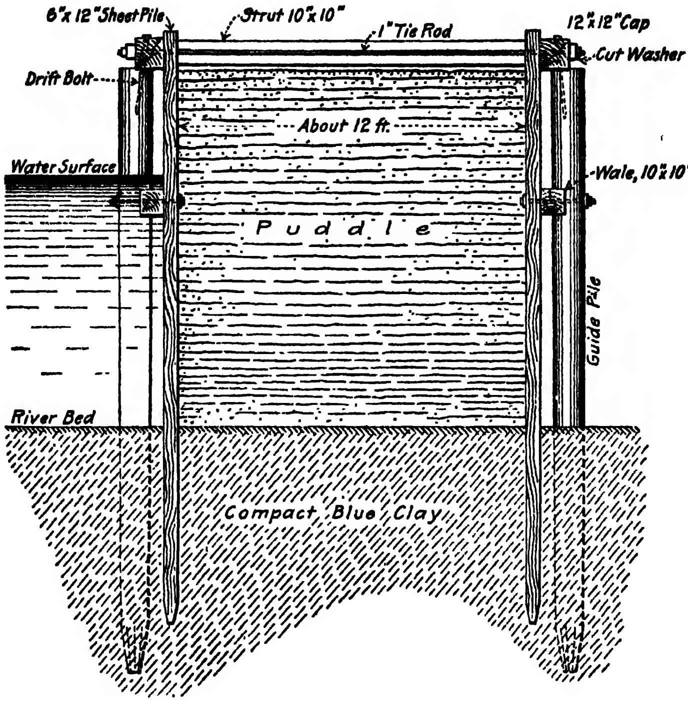


FIG. 68a.—Section of the Double Wall of a Cofferdam Showing Puddle Chamber.

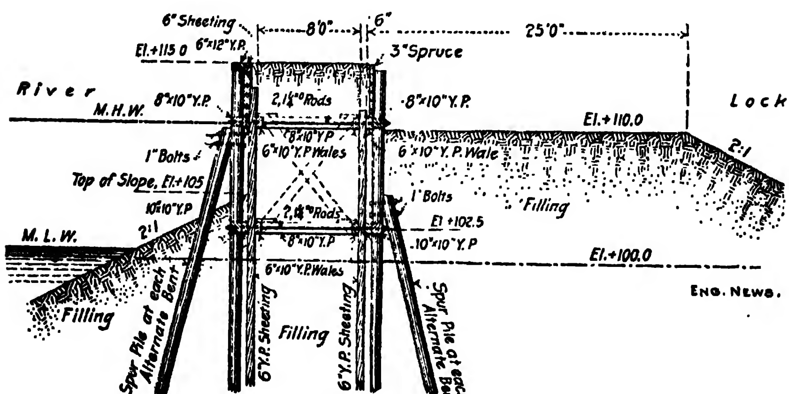


FIG. 68b.—Details of Double Wall of Sheet-Pile Cofferdam, Charles River, Boston, Mass.

the bottom to an impermeable stratum. This puddle filling, in addition to promoting water-tightness, will materially strengthen the structure. Clay is often banked around the outside of the cofferdam to safeguard it further against leakage.

The cofferdam should have its puddle chamber wide enough to develop the required strength, furnish water-tightness and afford sufficient space for placing machinery, gangways, etc. One rule for the width of unbraced cofferdams is to make it equal to the height above the ground up to 10 feet, and when the height is greater than this, make the width 10 feet plus one-third the height in excess of 10 feet. The design of sheet-piling is considered in Art. 65.

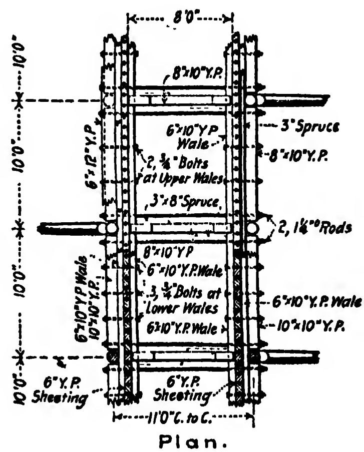


FIG. 68*c*.

A well-designed cofferdam of the double-wall type was used in the construction of the locks for the Charles River Dam, Boston, Mass., where the length was about 625 feet and the width about 250 feet, surrounding an area of approximately 4 acres. The maximum depth of water on the outside at low water was 20 feet. As shown in Fig. 68*b* and *c*, the cofferdam consisted of two rows of guide piles 11 feet apart, with piles spaced 10 feet on centers, which through wales supported 6-inch splined and grooved sheet-piling.

The guide piles were of spruce, 45 feet long, and each alternate pile was braced by a batter or spur pile. The sheet-piling was of yellow pine with spruce splines and was 38 feet in length. The remainder of the details are clearly shown in the diagrams. A filling of sand and clay was placed around both the inside and outside of the cofferdam as well as in the puddle chamber. On the inside it had a width of 25 feet at the top and then sloped down on a 2 on 1 slope, thus making virtually a combination pile and earth cofferdam. Although probably not an econom-

ical form of cofferdam for ordinary use, yet in a case like this, where the filling was permanent construction, it made an admirable structure to withstand the 37-foot head, which was approximately the maximum height of high water above the bottom of the lock masonry.

Considerable trouble was experienced with the type of cofferdam illustrated in Fig. 67*a*, and in building dam No. 19 on the Ohio River a different type was used. Guide piles about 25 feet long were driven 5 feet into the gravel bottom and spaced about 16 feet centers in the line of the cofferdam, the transverse distance being 18 feet. On these guide piles four lines of 6- by 8-inch waling pieces were spiked and vertical 2-inch sheathing was then fastened to the waling pieces after being driven slightly. The two sides of the 18-foot box thus formed were then tied together with cross-rods through the waling pieces, after which the box was filled with earth to form a puddle, the top of which was protected with a concrete slab. Earth was backfilled on either side of the box on a 1 to 2 slope.

ART. 69. SINGLE WALL WITH GUIDE PILES

Where the space available for the cofferdam is restricted or where the area of the site to be inclosed is small and the head of water not great, a cofferdam having a single wall is preferable to the double-wall type. Other conditions being the same, the former type will require more bracing than the latter, but in many cases this will prove cheaper than the extra wall.

Figures 69*a* and *b* show the details of cofferdams used for the rectangular and pivot piers for the Illinois Central Railroad bridge across the Tennessee River at Gilbertsville, Ky., both of which are standard types for single-wall cofferdams of moderate size. Before placing these cofferdams the bottom of the river was dredged down to about 17 feet below low water to hard gravel. Cofferdam guide piles were then driven and 10-by 12-inch wales bolted to the outside, after which 9- by 12-inch triple-lap sheet-piling was driven against the latter, penetrating the gravel from 4 to 6 feet. The piers were founded on bearing

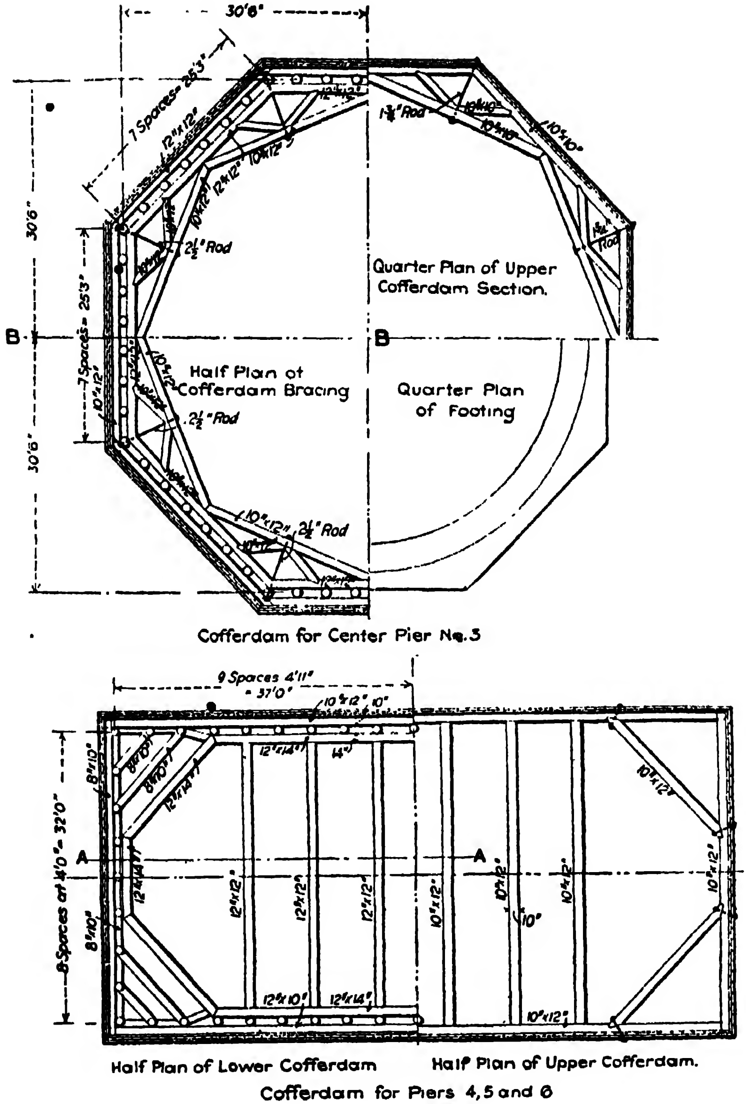


FIG. 69*a*.—Cofferdams with Single Walls of Timber Sheet-Piling Supported by Wales and Guide Piles, for Piers of Illinois Central Railroad Bridge over Tennessee River, at Gilbertsville, Ky. See also Fig. 64*a*.

piles driven from 16 to 20 feet into the gravel and cut off 2 feet above the bottom before the cofferdam was placed.

Before pumping out the water a 3-foot layer of concrete was placed on the bottom, thus preventing leakage of water beneath the cofferdam; later it served as a cap for the bearing piles. The bracing, which is clearly shown in the illustrations, was placed as the water was pumped out. The octagonal cofferdam

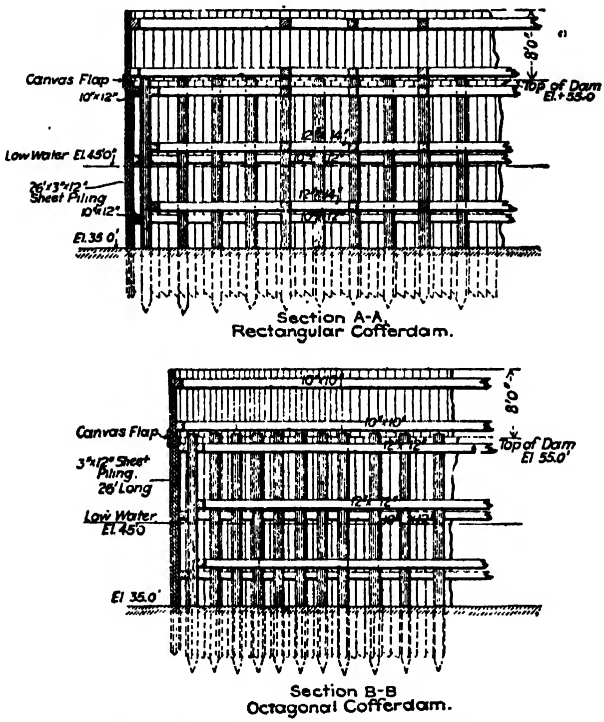


FIG. 69b.—Elevation of Cofferdam Walls.

was braced by annular trusses which, by their arch-like action, proved to be a very rigid form of bracing, and yet offered no obstruction to the work of building the piers, which were of concrete. The forms for these piers were braced against the trusses.

A good example of a very large and high single-wall sheet-pile cofferdam, very strongly braced, is illustrated in Fig. 69c,

this structure being used to found the pier of a lift bridge for the Chicago Terminal Transfer Railroad. Two sides of the

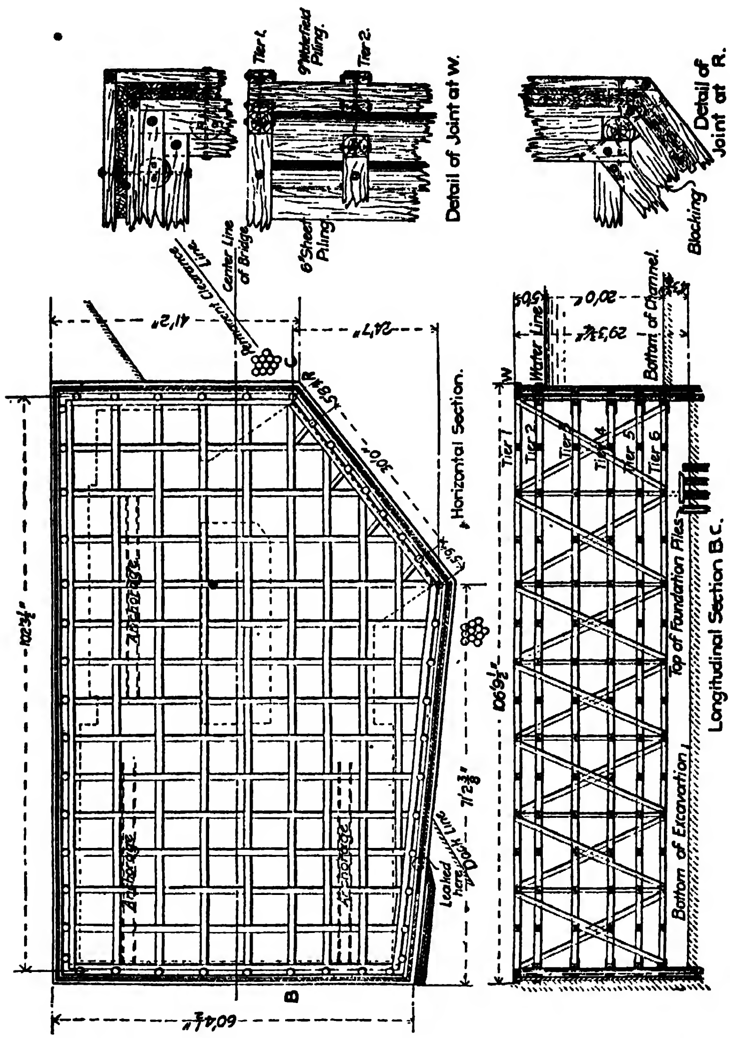


Fig. 69c.—Cofferdam for Substructure of Rolling Lift Bridge at Taylor Street, Chicago.

cofferdam were on land, one in water, and the other two partly in water and partly on land.

A row of guide piles, from 6 to 8 feet apart and 40 feet long, were first driven. [1]"Six tiers of inside and outside waling pieces were bolted to these piles, and on the land side 370 6- by 12-inch sheet piles 34 feet long were driven between the outer wales, and 6- by 12-inch horizontal guide pieces at the surface of the ground and 4 feet below it. On the water sides 274 34-foot Wakefield piles 9 inches thick, made of 3- by 12-inch planks, were driven in the same manner.

"The piles were driven as the excavation progressed inside of the cofferdam, and at the same time rows of transverse and longitudinal 12- by 12-inch horizontal braces, about 6 and 8 feet apart on centers and from 4 to 6 feet apart vertically, were set with the ends engaging the round piles on the center lines of the walls. At intersections these braces were supported on 8- by 8-inch vertical timbers; one of them was continuous and the other was cut to clear it, with the square ends abutting against the sides of the first piece and spliced across it with two side fish plates . . . The inside wales were of 12- by 12-inch timber (except in the upper two tiers, where 8- by 16-inch timber was used because it was conveniently available from the contractor's stock), all of them being lapped and halved at intersections. The outside wales were uniformly 6 by 12 inches. The round pile caps and the two upper rows of wales on the water side were made of 9- by 13- inch timber. All wales were bolted through the round piles, and the oblique joint in the Wakefield piling was tied by bolts through both faces.

"In the longest dimension of the cofferdam, the six tiers of horizontal struts in each longitudinal line were divided into seven panels by the vertical posts supporting them at the intersections of alternate transverse braces. Each panel thus formed on three of the long lines and one short line was X-braced with 2- by 10-inch planks, spiked to the longitudinal struts at all intersections and overlapping in the centers of the panels, as shown in the longitudinal sectional elevation. Six lines of similar bracing were provided for the transverse struts, but varied from that in the longitudinal direction in that the

[1] Engineering Record, vol. 50, p. 636, Nov. 26, 1904.

upper and lower pieces of the bracing overlapped each other by the width of the space between two transverse struts, thus increasing the amount of bracing and the rigidity at a point half way between the top and bottom of the cofferdam."

ART. 70. SHEET-PILING SUPPORTED BY FRAMES

Where the nature of the bottom is such that piles cannot penetrate the same, it is necessary to employ a frame to hold the sheet-piling in place. These frames are usually built on shore, floated to the site and sunk. Where piers are to be built under an existing bridge, it is sometimes possible to suspend the frame from the bridge.

SINGLE-WALL TYPE.—At the site of the bridge piers of the Chicago, Milwaukee, and St. Paul Railway near Kilbourn, Wis., only a few feet of sand covered the rock bottom on which the piers were to rest. As the channel was narrow and the current swift it was essential that the current be obstructed as little as possible, and for this reason the single-wall type was chosen in preference to that having a double wall. On account of the slight depth of sand, guide piles could not be used and so recourse was had to a frame. As shown in Fig. 70*a*, the cofferdam had V-shaped ends to diminish the force of the current against the structure and was held in place by wire guys anchored to the rocks on the sides of the river. The frame, the details of which are shown in the illustration, was sunk by weighting with scrap rails. The covering consisted of 9- by 12-inch Wakefield sheet-piling; in driving this piling care was taken to broom the lower ends to give a close fit to the irregular rock surface.

To aid in giving water-tightness to the structure canvas was placed around the outside of the cofferdam, and was so arranged that the lower part rested flat on the river bed for a distance of 8 feet out from the dam, while the upper part extended above water-level. The lower part of the canvas was first weighted down with iron rails and sand bags to make it fit closely, after which about 50 car loads of gravel were placed upon it. As

the water was pumped out, the structure was thoroughly braced as shown, but on building the pier this bracing was removed and the cofferdam walls braced against the pier.

One of the largest and highest cofferdams ever built of wood was of the single-wall sheet-pile-on-frame type, and was used for the Mare Island dry dock No. 2.[1]

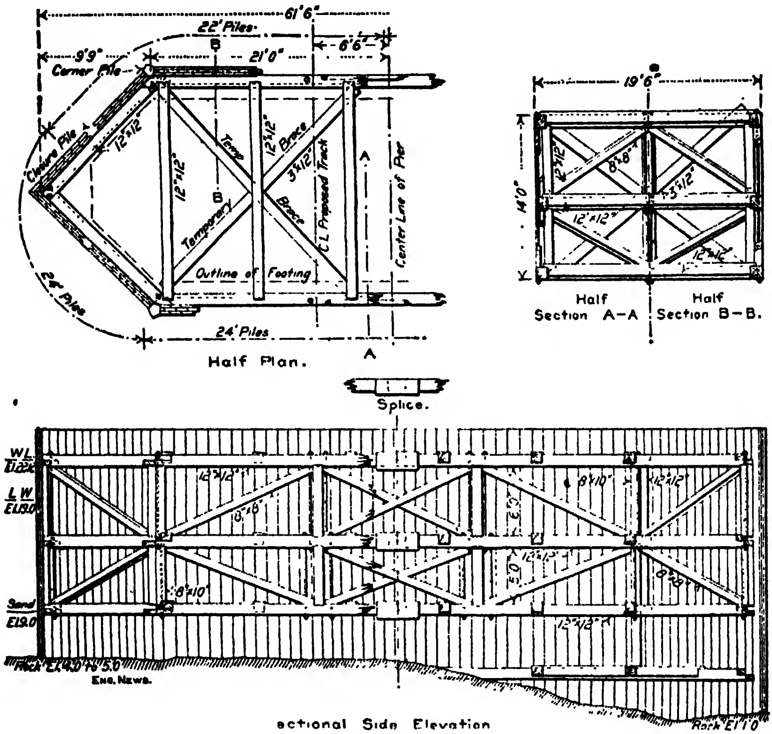


FIG. 70a.—Cofferdam for Pier of Chicago, Milwaukee, and St. Paul Railway, Kilbourn, Wis.

The cofferdam was approximately 150 by 800 feet in plan and the maximum head of water on it was 48 feet. The framework and bracing consisted of five horizontal courses of transverse and longitudinal timbers, the timbers of each course being connected to those of the adjacent courses by posts, the whole struc-

[1] For a complete description of this structure see the Engineering Record, vol. 57, page 428, Apr. 4, 1908.

ture being built as one unit which rested on bearing piles previously driven and sawed off under water. These longitudinal and transverse rows were 12 feet apart on centers. In the bottom course all timbers were 16 by 16 inches in section, while those in the next two courses were 14 by 14 inches, with 12- by 12-inch timbers for the two upper courses. The rangers, *i.e.*, the horizontal pieces forming the frame proper which holds the sheet-piling in position, were 20 by 24 inches in section for the bottom course and 12 by 12 inches for the top course, the other courses having intermediate sizes between these limits. The distance between courses was approximately 10 feet. In addition to the members mentioned, a large amount of bracing in both horizontal and vertical planes was used.

The sheet-piling units were formed of two 12- by 12-inch timbers fastened together side by side and were 60 feet long, this length being obtained by using two pieces, one 34 and the other 26 feet long. A tongue-and-groove joint was made by spiking to each piece of piling three 3- by 4-inch sticks, two on one side and one on the other, thus making each piling unit 30 inches wide. To give additional water-tightness to the cofferdam a large amount of filling was banked around the outside.

DOUBLE-WALL TYPE.—This form is little used, since it offers but slight advantages over the single-wall type and is considerably more expensive. It is more easily made water-tight than the single-wall form, but, on the other hand, it is very little stronger because strength is almost entirely dependent on the amount of internal bracing used. Where strength must be obtained without the use of bracing, the type described in Art. 71 should be used.

The cofferdams for one of the piers of the Chattahoochee River viaduct had an inside framework, 39 feet long by 15 feet wide, which was composed of horizontal frames of 6- by 8-inch pine timber braced with one set of longitudinal and two sets of transverse timbers. These frames were spaced from 2 feet center to center on the bottom to 3-foot centers at the top and were held in place by vertical posts between them, the total

height of the framework being 9 feet. The outside frames were sufficiently large for a 4-foot thickness of puddle and were connected to the inside frames by braces and rods. The framing was partly built on shore, launched, floated to place and there completed.

The bottom of the river had a seamy ledge covered with a layer of sand varying in depth from 6 inches to 3 feet. As soon as the framework was sunk, two rows of sheet-pilling, each row consisting of a double thickness of 2-inch pine plank, were driven, care being taken to break joints. The bottom of the puddle chamber was then covered with two layers of sacks loosely filled with sand, after which the remainder of the chamber was filled with clay puddle. Considerable trouble was caused by water coming up in the cofferdam through the seamy ledge and this leakage was stopped only after a 2-foot layer of concrete was deposited through the water and allowed to harden before pumping out the water.

Art. 71. Sheet-Piling Supported by Cribs

For cofferdams which rest on hard bottom and are too large to employ internal bracing economically, a series of cribs, laid up log-house fashion, are used to hold the sheet-piling in place. Each crib unit is made as long as can be conveniently handled and as wide as is necessary to develop the required stability. Rough logs are generally used, although in some cases they may be squared, but the latter offer only a slight advantage over the former. In building these cribs the bottom courses are usually started on land and the crib is built to a height sufficient to permit the top part being well out of water when it is first launched; after this it is launched, floated to place and completed. Where the stream is low at certain times of the year the cribs may sometimes be built in place. The bottom of each crib should be shaped to fit the rock bottom, and if a few feet of sand or other material overlies the bedrock this should be dredged out before placing the cribs. A part of the bottom of the crib is usually floored to permit placing stones so as to sink it.

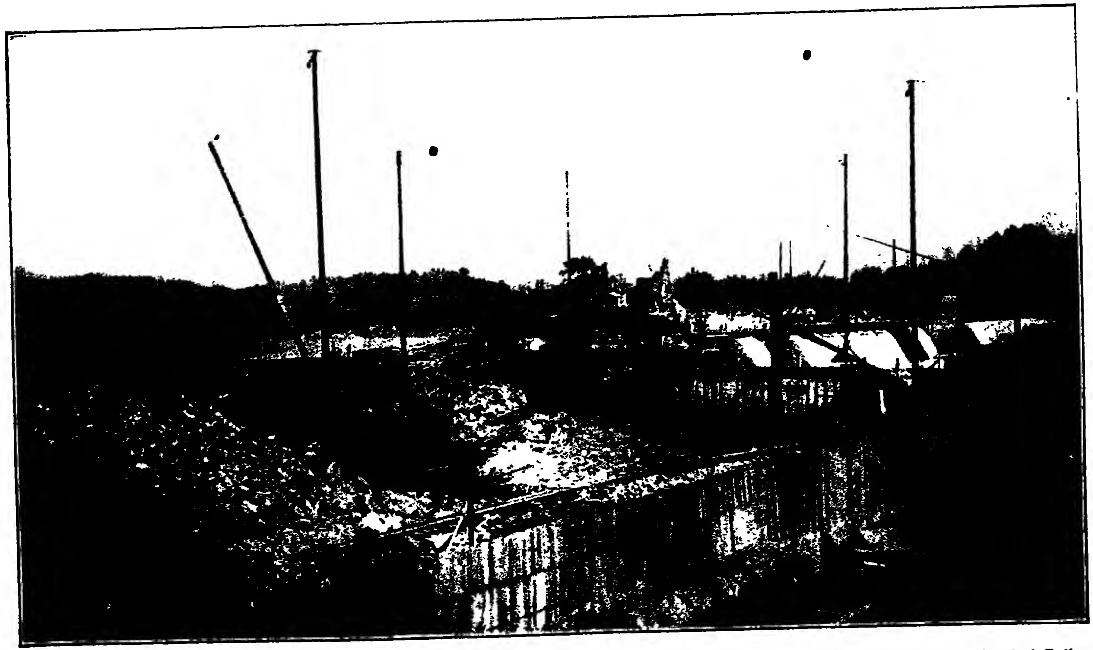


FIG. 71a.—Cofferdam of Connecticut River Power Company. Single Wall of Timber Sheet-Piling Supported by Loded Cribs. For additional views and details see Engineering Record, vol. 49, page 443, April 3, 1909.

(*Facing p. 226.*)

After all the cribs are sunk the remainder of the space inside of them may be filled with stones or earth. The latter material possesses the advantage of not only giving the cribs great stability but also to secure water-tightness. After the cribs are placed sheet piling is driven around the outside and banked with earth. This type of cofferdam is very widely used in building dams for hydroelectric plants.

Figure 71*a* shows a view of the cofferdam employed in the construction of a dam for the Connecticut River Power Co., near Vernon, Vt. The width varied with the height of the coffer-

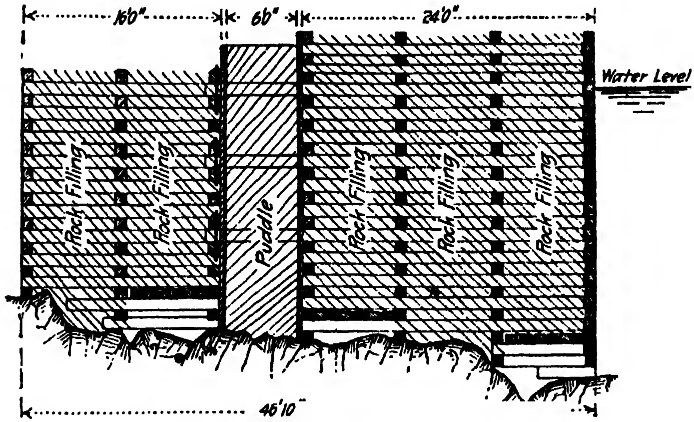


FIG. 71*b*.—Typical Section of Crib Cofferdam. Niagara Power Plant, Electrical Development Co. of Ontario.

dam; for the upstream one the maximum width was 35 feet, while the maximum height was 42 feet, or 16 feet above normal water-level. The structure was of the rock-filled type made of round logs in 7-foot checks, with the face logs slabbed on the sides to give good bearing for the sheet-piling. The top of the cribs were floored with logs to serve as a walk and also as a protection against ice pressures. On the outside the cribs were sheet-piled with 3-inch spline-and-grooved spruce, and this in turn was banked with earth up to normal water-level.

The cofferdams for the Niagara Power Plant of the Electric Development Co. of Ontario furnish an example of exceedingly

strong and rigid cofferdams placed under the most trying conditions. In some places the current had a velocity as high as 17 feet per second, which made it difficult to study the nature of the bottom and the depth of water previously to placing the cofferdams.

The widest part of the cofferdam consisted of two lines of parallel, rock-filled timber cribs with a space between, sheet-piled and filled with puddle as shown in Fig. 71*b*. Both cribs were built of squared timber with the outside wall of the outer crib laid solid. The width of the cribs varied to meet the variation in depth and the bottom of the cribs was made to fit the irregularities of the rock surface. In shallow water the cribs were built in place, but elsewhere they were constructed in the river upstream, and by means of cables from the shore they were floated into place and were sunk by filling with rocks the wells which had bottoms.[1]

ART. 72. STEEL SHEET-PILE COFFERDAMS

The advantages which steel sheet-piling possesses over the wooden type are discussed in Art. 62. On account of these advantages steel-piling is being used more and more in cofferdam work. The details of the structures differ but little from those using timber sheet-piling, the main difference being that the steel type, on account of the greater strength and positive interlock of the piling, requires less bracing.

Figure 72*a* indicates a good example of a steel sheet-pile cofferdam with guide piles. In the illustration, the guide piles and the outer course of wales are not shown, however. The bottom at the site of the pier consisted of hardpan to an unknown depth covered with about 6 inches of mud. The depth of water was about 9 feet at mean tide, which had a rise and fall of about 6 feet.[2] "Round wooden piles were driven 8 feet apart inclosing the site of the 83- by 15-foot

[1] For further details of this interesting cofferdam the reader is referred to the Engineering News, vol. 54, page 561, Nov. 30, 1905.

[2] Engineering Record, vol. 67, page 268, Mar. 8, 1913.

cofferdam; 6- by 12-inch inside waling pieces were bolted to them above high water.

"Spacing blocks 4 inches thick and 12- by 12-inch inside wales were bolted to the outside wales, forming guides, between which were driven a single row of Lackawanna 12-inch, 40-pound steel sheet piles 35 feet long. These were all assembled

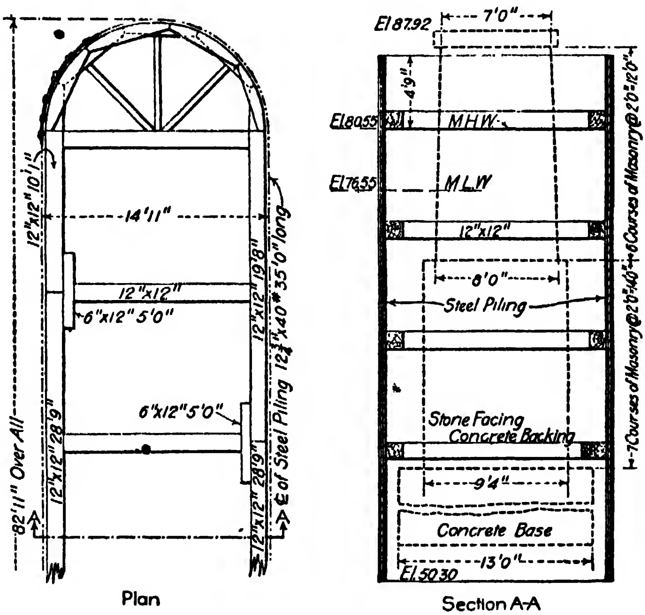


FIG. 72*a*.—Cofferdam for Highway Bridge Piers in Passaic River, at Bridge Street, Newark, N. J.

together before driving . . . and then driven . . . by one McKiernan-Terry steam-hammer weighing 5000 pounds and making about 225 strokes per minute. It was handled by the boom of a floating derrick and went round and round the cofferdam, driving each pile a foot or two at a time until the work was completed. The driving was very hard, many boulders being encountered, some of which were displaced and others broken by the piles. When they could be neither displaced nor

broken, driving on the piles that encountered them was discontinued, and adjacent piles were driven down to subgrade about 6 inches below the bottom of the footing.

"As the bottom was excavated inside the cofferdam, some of the boulders which obstructed the sheet piles were left in position and the sides of the excavation below them were closed as well as possible with bags of cement. The cofferdam resisted a pressure head of about 28 feet with very little leakage through the pile joints, which were packed with oakum . . . The long sides of the cofferdam are braced with 12- by 12-inch horizontal transverse struts 9 feet 7 inches apart on centers, in four tiers about 6 feet apart. At the rounded ends the inside waling pieces are made like arch centers of 12- by 12-inch double-scarf pieces, with radial braces to the middle of the adjacent cross-strut."

Some of the concrete piers for a bridge across the Illinois River at Peoria, Ill., were founded on bed rock 20 feet below the bottom of the river, where the depth of water was approximately 20 feet. To build these piers, cofferdams of steel sheet-piling on frames were used. By means of an orange-peel bucket the material of the river bottom was first dredged down to a layer of slate and soapstone, about 3 feet thick, which overlaid the rock. The excavation was made over a large area, so that the material overlying the slate would stand at its natural slope and still leave an area on the slate of sufficient size for the cofferdams, one of which was 39 by 40 feet in plan.

[1]"The steel-piling forming the sides and ends of the cofferdam was braced across the latter with five longitudinal and six transverse rows of 12- by 12-inch timbers to hold it in place when the water had been drawn down in the cofferdam. These timbers were placed in nine horizontal layers, varying from 2½ to 5 feet apart from the bottom to the top of the cofferdam. The horizontal layers were held apart by a vertical 12- by 12-inch timber at each intersection of the rows of braces. The timber crib formed by these braces and verticals was built in the water approximately over the site. The horizontal layer which

[1] Engineering Record, vol 55, page 247, Mar. 2, 1907.

would come at the level of the top of the slate and soapstone in the cofferdam was first assembled as a raft on which the verticals were erected and then the second horizontal layer was placed, sinking the crib thus formed to the water-level. The various horizontal layers were thus added in succession and when they had been completed the crib was towed over the site, sunk in position and anchored."

The steel-piling, of the Friestedt form, was driven around this framework through the slate and soapstone to rock, after which the material which had been previously dredged was backfilled around the cofferdam up to low water-level. After pumping out the cofferdam the layer of slate and soapstone was removed and the pier built.

Among the deepest cofferdams that have even been placed are those used in founding the piers of the Tunkhannock viaduct of the Delaware, Lackawanna and Western Railroad. These were land cofferdams and had a maximum depth of nearly 100 feet, with a depth of 65 feet below ground water-level. In principle, they closely resemble the method used in placing piers for buildings as described in Art. 127, and differ from the regular caisson, since excavation took place simultaneously with the driving of the sheet-piling, and since the lower part of the sheet-piling served as a form for the pier footing.

[1]"The cofferdam for pier 4 is typical of those of piers 3, 5, 6, 7 and 8 and was commenced by assembling on the surface of the ground a 43- by 49-foot rectangle made of 12- by 12-inch horizontal timbers spliced together to form one course of inner wales. Vertical posts were set up on this course and supported a second similar course about 16 feet above it, and two corresponding courses of exterior wales were erected outside of these and about 6 inches in the clear from them."

Lackawanna steel sheet-pile units 30 feet long were then placed between the outer and inner wales and driven by a steam-hammer going round and round the cofferdam, driving each pile unit 2 or 3 feet at a time. As the piling was

[1] Engineering Record, vol. 67 page 485, May 3, 1913.

driven the interior was excavated and the cofferdam braced with successive tiers of 12- by 12-inch longitudinal and transverse struts.

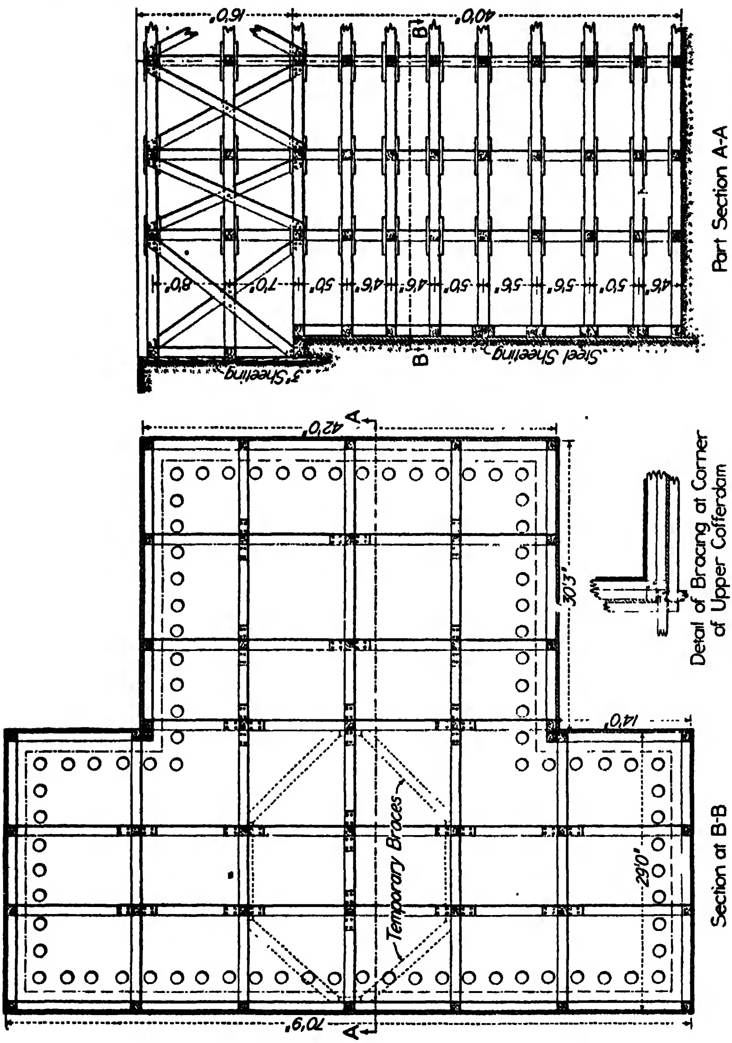


FIG. 72b.—Large Cofferdam for Abutment of Union Pacific Railroad Bridge over Kaw River, at Kansas City Kan.

After driving this set of piling to its full length, an exterior row, concentric with the inner row and 4 feet 8 inches beyond the same, was assembled and first driven to a penetration of

FIG. 72c.—Interior of Steel Sheet-Pile Cofferdam Showing Timber Bracing.

(Facing p. 232.)

about 12 to 15 feet. The space between the two rows was then excavated and at the same time the inner row was also driven, the upper tiers of bracing of the latter being transferred to the bottom and new sets of bracing furnished to the outer piling. In this way, by driving both outer and inner rows to their required positions, the excavation was carried to rock. The advantage of two rows of piling was in the easier driving thereby obtained. The lower part of the excavation was completely filled with concrete, the steel-piling serving as a form; the surface of the piling was protected from the concrete by tarred paper, thus permitting the piling to be withdrawn later.

Figure 72*b* illustrates a somewhat similar type of cofferdam used in the reconstruction of the Union Pacific Railroad bridge at Kansas City. The upper tier of sheet-piling was of wood. The details of the bracing are clearly shown in the illustration.

Figure 72*c* is a half-tone showing the details of the bracing used for the steel sheet-pile cofferdam at the Loomis Street tunnel, Chicago. The cofferdam was 75 by 53 feet in plan and the maximum head of water on it was about 53 feet. The bracing consisted of 12- by 12-inch timbers, spaced 8 feet apart horizontally and 4 feet vertically.

Few examples exist of the type of cofferdam consisting of steel sheet-piling on cribs. The reason for this lies in the fact that almost all the crib and sheet-pile cofferdams have been built in localities where timber is abundant and for this reason sheet-piling of wood is cheaper than that of steel.

Art. 73. Cellular Steel Sheet-Pile Cofferdams

The cellular steel sheet-pile cofferdam is a type that has been used with success in unwatering large areas. Water-tightness is obtained by the use of a double row of steel sheet-piling, interlocked and driven to rock, the space between the rows being filled with clay. Structural stability is furnished, for the most part, by riprap on the inside of the cofferdam.

The most notable example of this type is that used in the construction of a ship pier at the foot of West Forty-sixth Street, New York. It consisted of a core wall formed of cellular

pockets of steel-piling filled with earth, each pocket being about 16 feet wide and 24 feet long. The shore side of the wall was banked with riprap and the offshore side with earth, as shown in Fig. 73*a*. High water is 6 inches below the top of cofferdam.

The cells were made of Lackawanna piling of 37.2-pound section, each cell consisting of two slightly rounded longitudinal

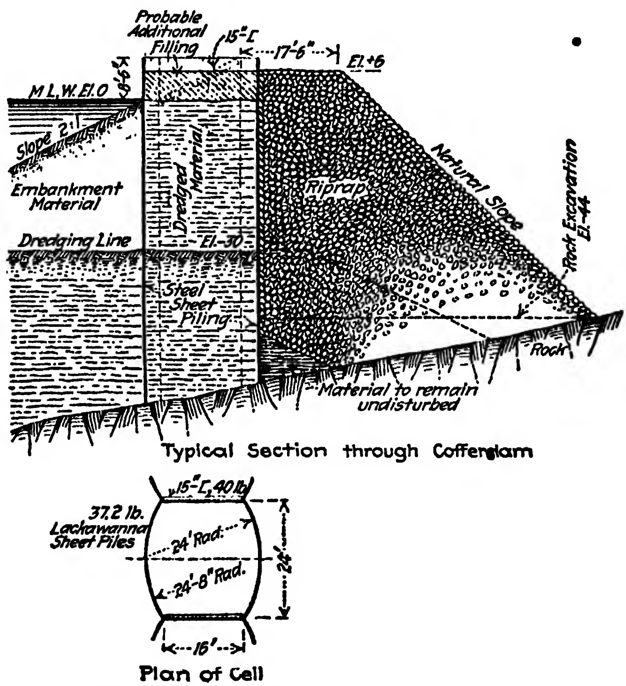


FIG. 73*a*.—Cellular Steel Sheet-Pile Cofferdam.

walls with transverse connecting straight walls, the latter being braced with heavy channels bolted to each section of the piling. All the piling was in single pieces, many of which were over 70 feet long.

The tension stress at the interlock of the piling may be analyzed as follows, it being a maximum when the cells are filled and no embankment around the outside. Assuming the weight of filling at 80 pounds per cubic foot and its natural

slope—river mud—at 3½ to 1, the following is obtained for a head of 58 feet by the Rankine formula for earth pressure:

$$p = wh\frac{(1 - \sin\varphi)}{1 + \sin\varphi)} = 80 \times 58 \times 0.57 = 2640,$$

where p denotes the pressure in pounds per square foot against the longitudinal walls at the bottom.

If the filling is assumed to act radially,

$$t = \frac{pR}{12} = 2640 \times \frac{24}{12} = 5280,$$

where t denotes the tension in pounds per lineal inch in the piling of the longitudinal walls and R the radius of curvature in feet. Likewise

$$t' = \frac{pd}{12} = 2600 \times \frac{24}{12} = 5280,$$

where t' denotes the tension in pounds per lineal inch in the piling of the transverse walls and d the spacing of the transverse walls in feet.

Tests made on the piling previous to driving showed a strength in the interlock of 9000 pounds per lineal inch.

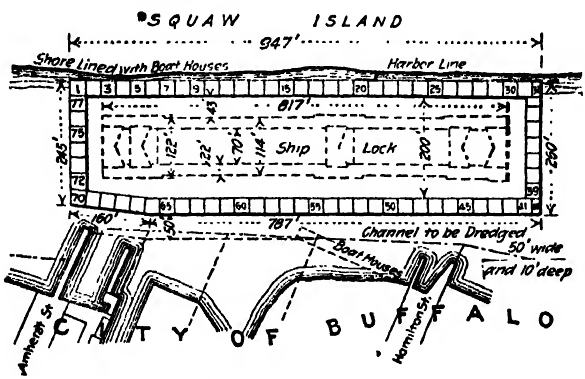


FIG. 73b.—Plan of Black Rock Cofferdam.

Previous to the construction of the New York structure the two most notable examples of this type of cofferdam were those used at Black Rock Harbor, Buffalo, and for raising the *Maine* in Havana Harbor, Cuba.

The Black Rock cofferdam was built to permit the construction of a ship lock, and was rectangular in plan, 260 by 947 feet overall, as shown in Fig. 73*b*. The depth of water at the site varied from 2 to 15 feet, averaging about 8 feet, while the solid rock on which the lock was built was about 40 feet below mean water-level. As shown in Fig. 73*c*, the sides of the cofferdam were made of two walls of steel sheet-piling, the space between the two walls being divided into pockets 30 feet square by transverse walls of the same piling as that used for the main walls, which served to connect the latter. A horizontal 15-inch, 40-pound channel was bolted to the tops of the piles of the inner wall and a similar channel was bolted at an inclination across the transverse walls as shown in Fig. 73*d*.

The piling was driven to rock and at first wooden guide piles and wales were used to maintain the alignment of the steel sheeting, but eventually these guides were dispensed with, the only ones used being 10- by 30-foot floating forms having one edge in the plane of the sheeting. The fine alignment attained by this simple method may be seen in Fig. 73*c*. After driving the piling, the pockets were filled with clay and to further strengthen the structure, as the inside was excavated, a bank of earth 25 feet high was maintained on the inside as shown in Fig. 73*d*. But in spite of this bank of earth the material in the pockets caused the inside wall to bulge badly between the cross-walls in both a horizontal and a vertical direction.

It is instructive to observe the plans of different pockets of the cofferdam, and the curvature of vertical sections after the steel sheet-piling adjusted itself to the pressure of the clay filling by developing tension in the interlock. Figure 73*e* gives the results of a careful survey of pocket No. 35 in which the maximum bulging of sides occurred. It should be noted how short a distance the bulging extended below the sand and gravel bank which was allowed to remain inside of the cofferdam. The diagram also shows vertical sections at the middle of pockets Nos. 30, 52 and 75, the relative location of the pockets being indicated in Fig. 73*b* (see also the half-tone view, Fig. 73*d*). Before this cofferdam was closed it was seen that the

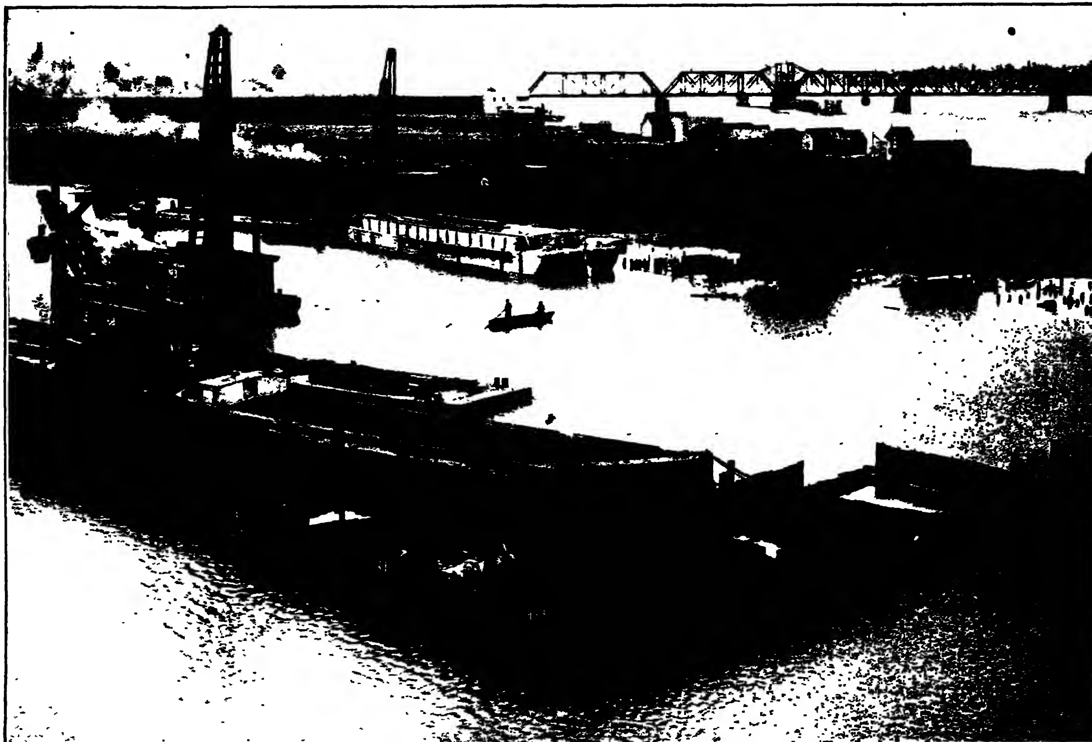


FIG. 73c.—Cofferdam for the U. S. Government Ship Lock at Black Harbor. Northeast Corner.

(Facing p. 236.)

FIG. 73*d*.—Cofferdam at Black Rock Harbor, Looking North from Southwest Corner. Shows bulging of pockets.

curved longitudinal walls made the better design and the last four pockets were constructed in this way.

In all, 6589 tons of steel sheet-piling were used in this cofferdam, there being 6870 linear feet of piling wall from 45 to 50 feet high. The price paid the contractor for building the cofferdam, which included furnishing all material, pumping out and maintaining the same, was $408,830. The type of piling used was that known as the Lackawanna, and which had

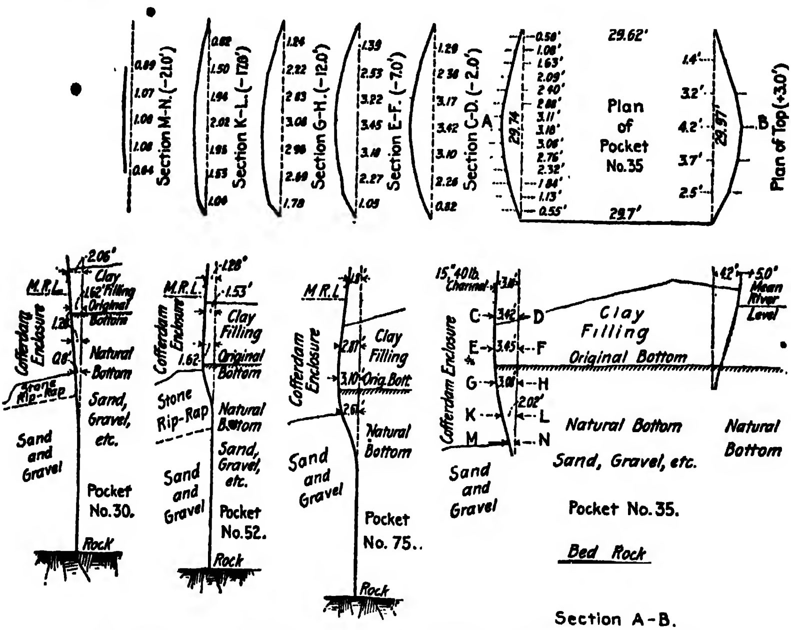


FIG. 73e.—Diagram Showing Deformation of Steel Sheet-Piling.

a web thickness of ½ inch and weighed 40 pounds per linear foot.

The cofferdam for raising the *Maine* represents a special type of steel cofferdam, very large and strong. [1]"The problem was to surround the wreck of the vessel, lying in about 29 to 37 feet of water, with a cofferdam, which when unwatered would be tight enough to prevent leakage, strong enough to resist outside water and mud pressures, and a protection that would

[1] Bulletin 102, Lackawanna Steel Co., Buffalo, N. Y.

assure safety during the work. The cofferdam should be self-sustaining, if possible. Bracing by struts across its interior to resist the water and mud pressures might be difficult to install and would interfere with the operation of removal. The borings indicated bad conditions for foundations. The building of a cofferdam without internal bracing, which would withstand pressures from a head of 37 feet of water and practically 21 to 23 feet of mud, was an unprecedented task.

"The cofferdam should be not only self-sustaining and safe against the pressures to which it was to be exposed, but it should also be capable of complete removal after it had served its purpose. It should be able to support more or less superimposed loads, for working platforms had to be built upon it. The work of unwatering the area inclosed had to be carried on from the top of the cofferdam; and afterward, men and materials had to be transferred from there to the interior, for work upon the wreck . . . The cofferdam decided upon consisted of 20 equal cylinders, 50 feet in diameter, and composed of steel piling 75 feet long . . ." (A plan is shown in Fig. 73*f*.)

"The length of the major axis of the cofferdam was practically 399 feet, and of the minor axis 219 feet, leaving a 20-foot clearance at the submerged bow of the ship and a 14-foot clearance at the stern, with 45 feet at the side cylinders. Such clearance was necessary to avoid portions of the wreck which had been blown beyond the position occupied by the hull.

"The units of the cofferdam were made cylindrical for the reason that the extremely high pressures, which would be exerted by the mud filling, would act radially and uniformly on each pile, straining each joint to the same amount at equal depths, and in the entire cofferdam cylinders would deform least from play in the piling interlocks."

The piling used was the Lackawanna section, weighing 35 pounds per linear foot, and had a web ½ inch thick. The piles were driven so that their tops were 2 or 3 feet above normal water-level (Fig. 73*h*) and the 75-foot length of piling, which penetrated the harbor bottom to a distance of approximately 35 feet, was made of two lengths spliced together with channels.

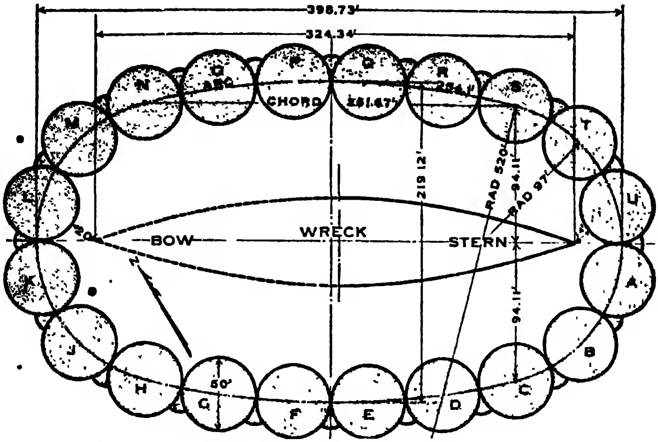


FIG. 73*f*.—Plan of Cofferdam for Raising the *Maine*.

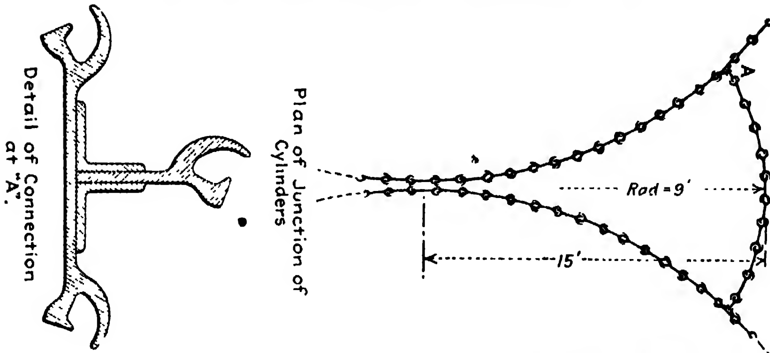


FIG. 73*g*.—Connection of Cofferdam Cylinders.

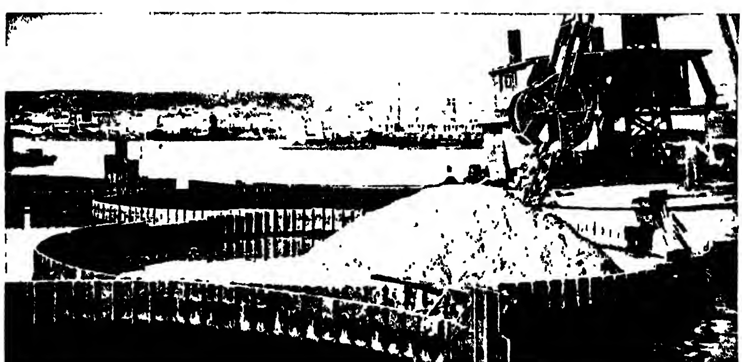


FIG. 73*h*.—Filling Clay into Cylinder *A*. Part of *B* in Foreground.

The cylinders were driven tangent to one another and a curved diaphragm of steel sheet-piling, shown in Fig. 73g, was driven to connect adjacent cylinders. It was planned to fill the cylinders with heavy clay, it being thought that this clay would displace the 25 feet of harbor silt in the cylinders. As time did not permit the use of dipper dredges for this work, hydraulic dredges were used and as a result the new material blanketed the existing harbor silt in the cylinders. When the cofferdam was ready to be unwatered, it was found impossible to solidify this silt and it remained in a semi-fluid condition throughout the work.

As the water was pumped out inside the cofferdam, the cylinders became badly distorted, taking an elliptical form

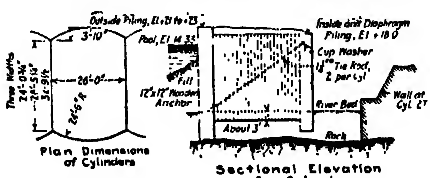


FIG. 73*i*.—Cellular Cofferdam at Troy, N. Y.

and distorting inward. To prevent failure, riprap was placed against the inside of the cofferdam and the filling of the cylinders was shifted somewhat to form approximately a continuation of the riprap slope. Before the cofferdam could be completely unwatered, a number of heavy braces had to be set between the cylinders and the wreckage of the *Maine*.

In the above three examples stability was due, at least in part, to heavy riprapping on the inside. In the construction of a cofferdam at Troy, N. Y. the structure, shown in Fig. 73*i*, was self-supporting. A hydrostatic head was assumed on the outside from rock to the top of the pockets and this force was balanced solely by the weight of the fill assumed at 110 pounds per cubic foot for the upper half and 65 for the lower half. As proportioned by these assumptions the resultant pressure came a little within the middle third. Assuming a coefficient of friction of 0.5, the structure was just safe against sliding.

ART. 74. CRIB COFFERDAMS

Where the cofferdam is to rest on bed rock which is approximately smooth and level, a crib cofferdam, formed with one or two walls of squared horizontal timbers laid closely, may be used in place of the sheet-pile cofferdam. Where the single-wall type is used it is ordinarily made an integral and permanent part of the pier, and as such is not a cofferdam, but a caisson. For a description of this type see Art. 86.

In his book on Sub-aqueous Foundations, FOWLER describes a double-wall crib cofferdam used by the Chicago, Burlington and Quincy Railroad, which was made from 2- by 8-inch and 2- by 10-inch fence boards laid flat. The two walls were thoroughly tied together and the space between filled with puddle.

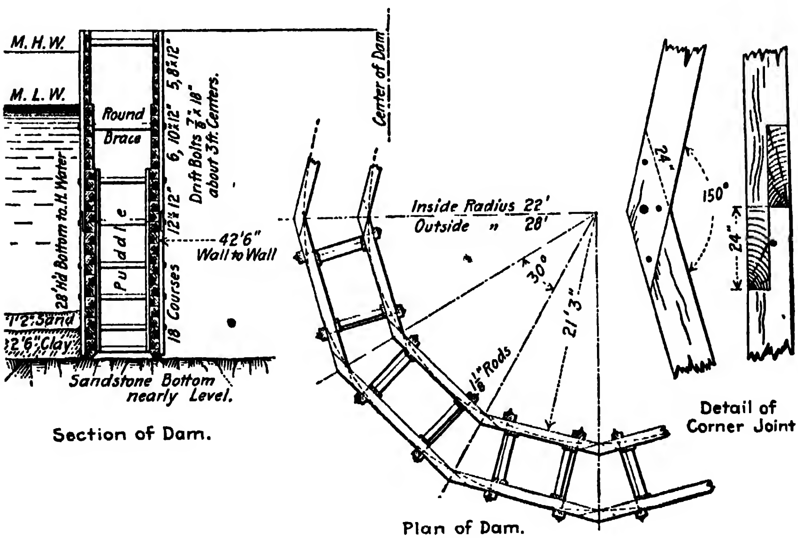


FIG. 74*a*.—Cofferdam for Pivot Pier of Arthur Kill Bridge.

Figure 74*a* shows a polygonal cofferdam of the crib type which was used for the center pier of the Arthur Kill bridge. At the site of the cofferdam the depth of water at high tide was about 28 feet, with about 4 feet of mud and clay overlying bed

rock. This mud and clay was dredged out previously to placing the cofferdam. The latter had 12 sides with walls 4 feet apart in the clear, and in this space puddle was dumped. All courses of timber were thoroughly drift-bolted together and all joints caulked with cotton wicking. No internal bracing was used. Before pumping out the water a 4-foot layer of concrete was deposited all over the bottom and allowed to harden for a week.

The cofferdam for the new inlet tower of the St. Louis waterworks was of the double-wall crib type, 38 by 76 feet in plan and 22 feet high. The walls were composed of horizontal 12- by 12-inch material and were 3 feet apart in the clear. The joints between all courses were carefully calked. The cofferdam was braced transversely by three vertical rows of horizontal 12- by 12-inch timbers spaced 4 feet apart vertically, and extending from outside wall to outside wall, thus tying the walls together as well as bracing the cofferdam. The ends were braced by similar horizontal 12- by 12-inch diagonal timbers, running at an angle of about 45 degrees from the center of the ends to the sides.

The river bottom was bed rock and the depth of water about 15 feet, the current having a velocity of from 6 to 8 miles an hour. The cofferdam was held in place by three triangular cribs filled with rocks and sunk upstream from the cofferdam and tied to the latter by cables. The puddle chamber was partly filled with concrete in sacks and puddle placed on top. Sacks of clay were also banked around the outside.

Cofferdams are widely used as temporary adjuncts to open and pneumatic caissons, but as the details differ widely from the types described in this chapter and resemble closely the caissons themselves, they will be described in the chapters dealing with such caissons.

ART. 75. MOVABLE COFFERDAMS

Unless it forms an obstruction to navigation, only that part of the cofferdam above low water is sometimes removed, because the salvage value of the material is less than the cost of getting it out, except where steel sheet-piling is used.

Where the same size and style of cofferdam is to be used for a number of piers it will often prove advantageous so to construct a cofferdam that it can be used over and over again. In one type, that of the cofferdam on grillage, it is so easy to make its sides removable that it is universally done, even though they may not be used a second time.

A movable cofferdam consisting of sheet-piling supported by a crib was used in constructing the piers of the Falls-of-Schuylkill bridge, of the Philadelphia and Reading Railroad. When in position, the cofferdam was 62 feet long, 36 feet wide and 16 feet high. The cribs were 10 feet thick, making the inside dimensions 42 by 16 feet. The cofferdam was divided vertically through each short side into two parts of equal size and these were floated separately to the site, joined together and sunk. Each section had water-tight compartments to assist in floating and these were filled with water and stone, while other non-water-tight compartments were filled with stone, when it was desired to sink the sections. On reaching the rock bottom, sheet-piling of jointed planks, 3 or 4 inches thick, was placed on the outside and spiked there. Puddle was then placed around the outside, after which the cofferdam was pumped out. Two sets of horizontal bracing connecting the long sides were placed as the water was removed.

In placing cylinder piers for the Queen's bridge, Melbourne, Australia, square movable cofferdams of the sheet-pile-on-frame type were used. One side opened outward as a door, thus permitting the cofferdam to be removed on completion of a pier. [1]"The dam was built on shore complete, and launched ready for immediate use on the site of a cylinder. The sheet-piling was vertical and consisted of 12- by 4-inch rough-sawn Oregon planks, supported by horizontal frames of 12- by 12-inch Oregon timber, spaced close together near the bottom of the river, to carry the greater pressure of water. Up the four corners of the dam were 12- by 12-inch Oregon timbers, into which the frames were checked and by which they were kept to their proper spacing, and which formed supports for the door. Outside

[1] Engineering News, vol. 33, page 230, Apr. 4, 1895.

the sheet-piling, at the top and bottom frames, there were outside wales, 12 by 6 inches (keeping the sheet-piling in place), bolted to the frames inside by 1-inch bolts, two to each wale, passing between two sheet piles." The sheet-piling was flush with the bottom of the frame and extended a few feet above the top.

At the site of the piers there was about 3 feet of soft silt covering the rock. This silt was covered with puddle before placing the cofferdam. After sinking it by weighting, the sheet-

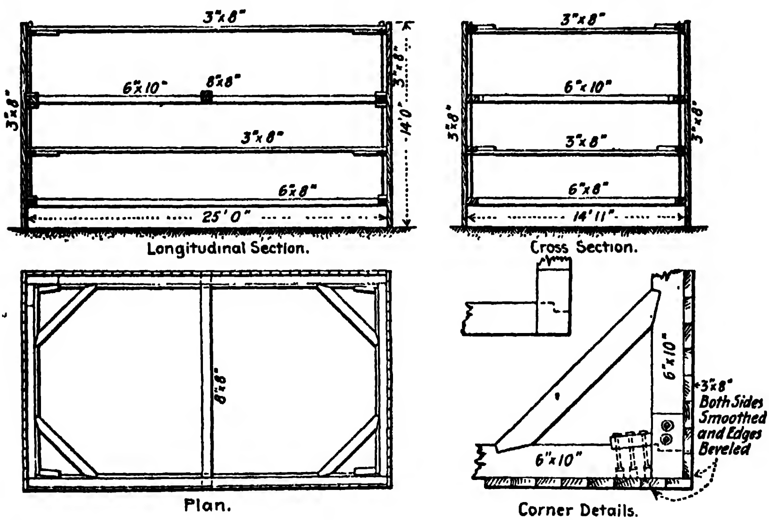


FIG. 75a.—Cofferdam Used on Key West Extension of Florida East Coast Railway.

piling was driven through the puddle and silt. On pumping out the cofferdam much of the silt ran into the interior and the clay took its place, thus sealing the structure. To remove the cofferdam the sheet-piling was first drawn up, the loading taken off, the door opened and the cofferdam floated out. At first tarpaulin was placed around the outside of the cofferdam but it was later found that this was unnecessary, since the sheet-piling was water-tight without it.

Figure 75a illustrates the form of a movable cofferdam used in constructing the piers of the Key West Extension of the Florida

East Coast Railway, where the depth of water did not exceed 8 feet. The two sides and the two ends formed independent portable sections which were connected together by means of 1¼-inch vertical rods running down through the overlapping rangers at the corners of the cofferdam.

At the site of the piers sand overlaid the coral rock. Piles, for the foundation of the pier, were first driven until the tops were 2 feet below low water, after which the cofferdam was

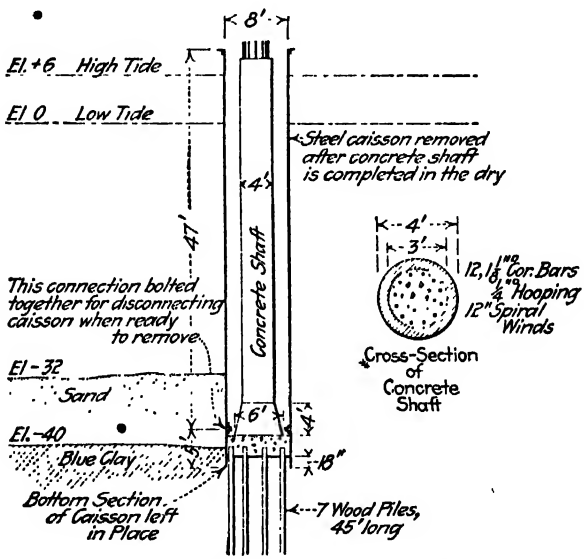


FIG. 75*b*.—Removable Cylinder Cofferdam.

assembled on a barge, lifted from the same and set in place. The sand was then pumped out by a centrifugal pump, after which a 2-foot seal of concrete was placed over the whole bottom. After allowing this concrete to harden for seven days the cofferdam was pumped out, forms placed and the pier built. On completion of the pier the rods were withdrawn. which allowed the sections to float free.

A novel type of cofferdam, shown in Fig. 75*b*, was used in Charleston, S. C., in which to build 4-foot concrete cylinder shafts. An 8-foot steel cylinder of ⅜-inch plate, the lower 5 feet of which was attached to the part above by a butt joint,

was driven into the river bed and the inside excavated with an orange-peel bucket to a short distance above the cutting edge. Foundation piles were then driven, the bottom sealed with concrete and the concrete shaft built up in forms, after which all of the cylinder except the lower 5-foot section was moved to another location.

After excavating with the orange-peel bucket, the water was pumped out and the bottom leveled by hand digging. In some cases the piles were driven in the dry and in others through

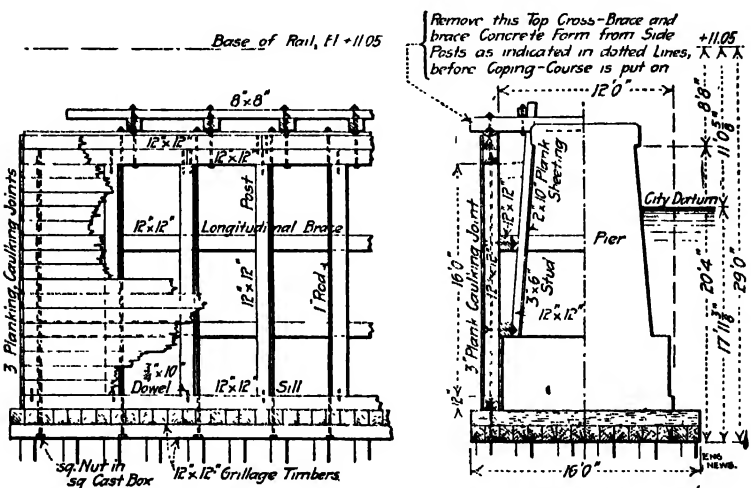


FIG. 75c.—Cofferdam for Rest Pier at Chicago and Northwestern Railway Lift Bridge at Kinzie Street, Chicago.

the water. On completion of pile driving, the cylinder was again pumped out, the piles sawed off and the 3-foot bottom course of concrete placed in the dry.

MOVABLE COFFERDAMS ON GRILLAGE.—On account of its convenience and ease of manipulation a movable cofferdam is almost universally employed where a timber grillage foundation on piles is used for the pier. The grillage and the cofferdam form an open box constructed on shore or on a barge or raft, launched, floated to the site and sunk on the pile foundation by building the pier in the box. This type differs from the box

caisson, described in Art. 83, since the sides of the former are not a permanent part of the pier. After the pier is built to above high-water level the cofferdam is removed, the sides being so fastened to the grillage that this can easily be done.

Figure 75c shows the details of the movable cofferdam used for the 12 by 41½-foot pier of the Kinzie Street drawbridge in Chicago. This cofferdam was connected to the grillage by 28 vertical 1-inch rods, 21½ feet long. To sink the structure, concrete forming the pier was placed in the same, the cofferdam itself serving as a form for the concrete up to an elevation shown in the drawing, and above this regular forms were used. On completion of the pier the rods were removed, which permitted the removal of the cofferdam from the grillage.

A very simple cofferdam on grillage was used in building the foundation piers of the Bellevue Hospital boiler house, a section of which may be seen in Fig. 75*d*. The largest was approximately 14 by 52 feet in plan and 12½ feet high. The most interesting feature is the very thin grillage used, it being composed of two crossed courses of 2-inch tongue-and-grooved planks. It was desired to use a thickness which would give enough strength for launching and sinking stresses, and yet be sufficiently flexible so that a uniform bearing over the slightly irregular pile tops would be secured.

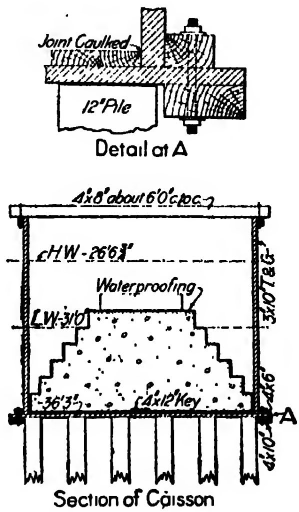


FIG. 75*d*.—Cofferdam with Removable Sides.

MOVABLE COFFERDAMS ON CONCRETE.—The cofferdams for the Mystic River bridge of the Boston Elevated Railway had their lower parts composed of concrete scows about 4 feet high and varying in size from 16½ by 44 to 19½ by 124½ feet. The bottom was 6 inches thick and the sides 12 inches, all concrete being 1-2-4 and well reinforced. Cross-partitions 9 inches thick were placed at 9-foot intervals. Around the

bottom, projected a flange 12 inches thick with eyebolts cast in to hold the wooden sides in place.

These sides were of two sections, the lower one 16 and the upper one 17 feet high, and were composed of 10- by 10- or 10- by 12-inch timber laid solid and sheathed on the outside with 2- or 3-inch material. The edges of the sheathing were beveled to provide for calking with oakum. The cofferdams were built on ways, launched, towed to the site and sunk on pile foundations previously prepared by dredging and driving piles, the tops of which were cut off by machine. The concrete

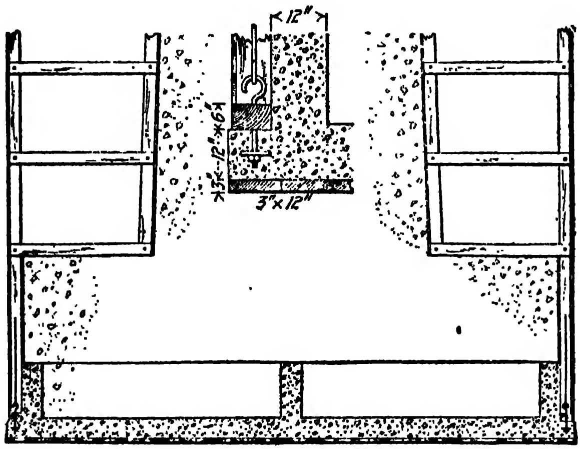


FIG. 75*e*.—Movable Cofferdam on Concrete for Bridge at Portland, Me.

bottom was cast on a layer of 3-inch sheathing which was a permanent part of the structure. The tops of the piles were 30 to 40 feet below mean high water.

The concrete piers were built in forms in the cofferdams, sinking being effected by the weight of the concrete. On completion of sinking the cofferdam sides were removed and used again.

Figure 75*e* illustrates the same type of construction as used for a bridge in Portland, Maine. Here the bottom, sides and interior walls had a thickness of 12 inches, the box being 4½ feet high. The sides of the cofferdam were composed of

studding and calked sheathing. The largest cofferdam was $32\frac{3}{8}$ by $88\frac{1}{8}$ feet.

Art. 76. Miscellaneous Types

The foregoing articles have dealt with what may be called standard types of cofferdams, but many cofferdams have been constructed which are either a combination of any two standard types or which differ fundamentally from those which have been described. For instance, in the cofferdam for the Dearborn Street bridge, Chicago, a double-wall sheet-pile cofferdam was used, the outer wall being composed of Wakefield sheet-piling and the inner wall of Friestedt steel sheet-piling, thus giving a composite wood and steel sheet-pile cofferdam.

Figure 76*a* shows a form of cofferdam known as the A-frame type which is used on bedrock. This particular one was used on the New York Barge Canal and consisted of a series of bents, spaced 6 feet center to center. On these bents rested purlins, which in turn supported the sheathing of jointed and calked 3-inch planking. As shown in the illustration, the structure was braced against sliding by having certain of the struts bear against concrete footings on the rock. This form of bearing can be made only when the rock is exposed at times. The maximum head of water supported was 18 feet.

Another cofferdam of the same type was used for some canal work at Keokuk, Iowa. Here it was necessary to construct the cofferdam without drawing off the water in the canal, and hence it was built away from the site, brought to place, and sunk by weighting with iron rails. Water-tightness was promoted by covering the sheathing with canvas.

In constructing concrete wharves at Fort Mason, San Francisco, steel-cylinder cofferdams, 7 feet 7 inches in diameter and 50 feet long, were used in which to construct reinforced-concrete piers. These cylinders, weighing 17 tons each, were driven by a pile-driver through the bottom and into hardpan, after which the water was bailed out, the mud removed, wooden forms placed and the 4-foot reinforced-concrete piers with

enlarged bases, 6½ feet in diameter, cast. After the concrete had set, the cylinders were pulled by the pile-driver, the required pull being about 50 tons.

In Volume 26 of Revue Technique, the proposed design for a cofferdam of ice to close the entrance to the outer basin at the Port of La Rochelle is described. The freezing was to be done with refrigerating machines, sheathings of non-conducting material being placed between the water to be frozen and that

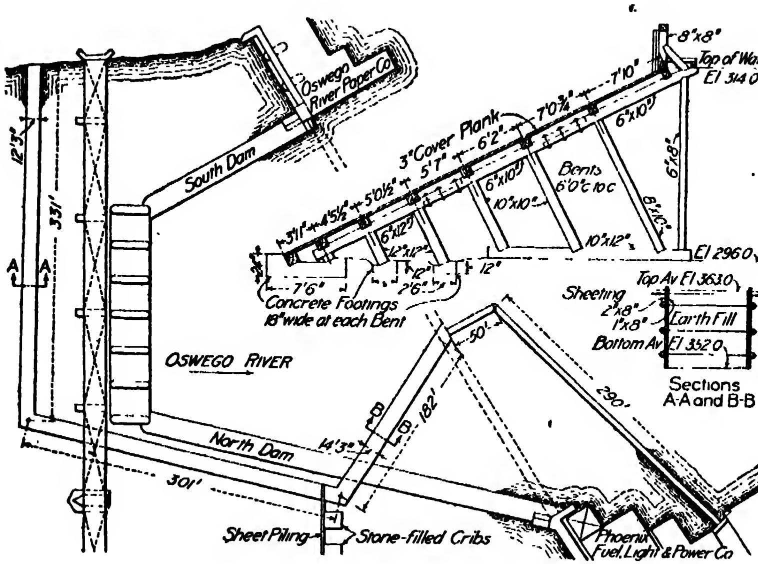


FIG. 76a.—A-Frame Cofferdam Used on New York State Barge Canal.

to be left unfrozen. It was estimated that such a cofferdam would cost less than any ordinary form.

Another proposed type described in the same article was of reinforced concrete. It was designed with inclined sides, the width of the base being 19.7 feet, the width of the top 4.9 feet and the thickness of the walls about 10 inches. The walls were to be well braced by struts. The structure was to be built away from the site, floated to place and sunk by filling with clay. It was estimated that it would have sufficient stability to withstand a head of 24.6 feet of water.

ART. 77. PUDDLE AND LEAKAGE

The construction of a water-tight cofferdam, is seldom attempted, for the cost of such is usually prohibitive. The greatest trouble from leakage occurs in the case of cofferdams which rest on rock, for here it is almost impossible to prevent the water from running in between the bottom of the cofferdam and the rock. But if the structure rests on clay and the sheet-piling is driven well down, there will be but a slight amount of leakage at this point. Where the leakage occurs through seams in the rock it may be stopped by filling the seams with grout pumped in through pipes about 4 inches in diameter. Another method is to dump clay, sand, ashes, etc., all around the cofferdam with a view of shutting off the water-supply of the crevices.

When the leakage is due to irregularities in the rock surface, concrete in bags may be placed on the bottom, or water-logged oat straw may be sunk by mixing it with ashes or covering it with a wire net loaded with sand and clay, after which the rest of the puddle filling may be placed. Another method of preventing leakage on a rock bottom is to use canvas as noted in previous articles.

Leaks often develop in double-wall cofferdams by the filling between the walls not compacting well, or settling after being placed and leaving openings beneath cross-braces. To compact this filling, piles are sometimes driven into it or stock ramming may be resorted to. The latter consists of forcing clay cylinders through pipes into the filling. Another method sometimes used is to drive a hole in the puddle above the leak and ram in quantities of excelsior. This excelsior swells quickly on getting wet and also acts as a filter.

With single-wall sheet-piling the joints above ground level often leak excessively. This may be prevented by putting down outside of each joint a V-shaped wood trough consisting of two boards nailed edge to edge and filling the space between with puddle.

The best puddle for a cofferdam is a mixture of clay and sand or clay and gravel. The clay should be cohesive and

impermeable to water. The cohesive quality may be tested by working it with a small quantity of water and then forming it into a cylinder 1½ inches in diameter and 10 inches long. If when suspended at one end while wet it does not break, it has sufficient cohesion. To test the degree of impermeability a considerable quantity should be worked into a plastic mass, hollowed out, and filled with water. The clay should hold this water for some time.

In making puddle only enough sand should be added to the clay to prevent cracking by shrinkage in drying. Puddle should be placed in layers about 3 inches thick and well chopped with spades, water being added at the same time. The spade should pass through two layers to insure bonding the layers together. For complete specifications for clay puddle the reader should consult BRYNE's Inspectors' Pocket-Book, third edition, page 272.[1]

ART. 78. DESIGN OF COFFERDAMS

Like most structures used in foundations, a purely theoretical design of cofferdams leads to unsatisfactory results. For some types it is a simple matter to design the structure to resist the hydrostatic pressure, but to design it properly to resist safely the pressure of the earth filling, or of freshets, ice or floating logs, requires much experience.

Earth cofferdams usually fail by the water seeping through and enlarging a channel until a washout takes place. For this reason such cofferdams should be carefully watched to detect small leaks that they may be checked quickly after starting. In general, if the cofferdam is made of a good mixture of clay and sand, has a width of at least 3 feet at the top, which is well above high water, and has sides inclined at the natural slope of the material, the cofferdam will be safe.

In the single-wall sheet-pile cofferdam with guide piles, if the wales are at the top and bottom, the sheet piles may be assumed to act as simple beams, with a load per vertical foot varying

[1] John Wiley & Sons, New York.

uniformly from zero at the water surface to a value of wd pounds per square foot at the surface of the earth, where w is the weight in pounds of a cubic foot of water and d the depth of the water in feet (for a discussion of the design of sheet-piling see Art. 65). The wales take the reactions of the sheet-piling and transfer them as supported, partially continuous, or continuous, beams, to the guide piles. Conservative engineers usually design the wales as simple or supported beams. Formulas are given in Art. 79 for the spacing of wales. If no bracing is used, the guide piles should be designed as cantilever beams with loads coming from the waling pieces. The maximum moment will occur at or below the mud line. If firmly braced at the top by struts extending across the cofferdam, it will be best to design the guide piles as simple beams.

Each wall of the double-wall sheet-pile cofferdam with guide piles may be designed somewhat in accordance with the above outline. The outer wall will be subjected to water pressure from the outside and earth pressure from the inside. Experience shows that usually the pressure from the puddle will, for equal heads, be larger than the pressure from the water. This will cause a stress in the tie rods connecting the two walls. The inner wall must be designed to resist the forces due to the puddle filling.

In the design of a cofferdam composed of sheet-piling on frames and the corresponding bracing, the outside pressure is the only force to be considered. The sheet piling acts as a beam between horizontal rangers, and the latter act as beams between bracing struts. In Art. 79 is an example of this type of design.

The sheet-pile-on-crib cofferdam must be designed so that the cribs will not overturn or slide. To be safe against sliding, the weight of the cribs and the filling per linear foot of length, multiplied by the coefficient of friction between the crib and rock, must be greater than $wd^2/2$, where the terms have the same meaning as those previously given. To be safe against overturning the weight per linear foot of length, including filling, multiplied by one-half the width, must be greater than $wd^3/6$.

Before attempting to design a cofferdam the literature on the subject should be carefully read, for, as stated in the first part of the article, no purely theoretical design will result in a thoroughly satisfactory structure. In the preceding articles, standard types and standard methods of construction are described and a careful reading of this material will help the inexperienced engineer. For more detailed information the reader is referred to the carefully selected list of references in Chap. XIX.

ART. 79. EXAMPLE OF COFFERDAM DESIGN

To get the vertical spacing of waling pieces of equal strength for the type of cofferdam shown in Fig. 79*a*, the following formula[1] may be used:

$$D_n = 0.314 \frac{k^{1/2} b^{1/2} d}{p^{1/2} s} \left[N^{3/2} - (N - 1)^{3/2}\right], \tag{1}$$

in which p denotes the equivalent unit fluid pressure in pounds per square foot, s the span of the wales in feet, k the allowable unit-stress in pounds per square inch of timber in bending, b the width in inches of the wales acting as beams, d the depth of same, D the distance in feet from the surface to the wale in question, and N the number of rows of wales from the surface not counting the top row.

This formula is developed on the basis that the top waling takes no load, the load from the areas A, B, C, etc. going respectively to the waling sets 1, 2, 3, etc. Since the wales are of equal strength, these areas will be equal. If W denotes the allowable load on each wale acting as a simple beam and h the total depth in feet,

$$\frac{ph^2 s}{2} = NW = \frac{Nkbd^2}{9s}. \tag{2}$$

Taking moments about the top and equating the moment of the external pressure to the moment of the loads on the wales,

[1] See Engineering News-Record, vol. 82, page 708, Apr. 10, 1919.

$$\frac{2NWh}{3} = W(D_1 + D_2 + D_3 \ldots + D_{n-1} + D_n) \quad (3)$$

$$D_n = \frac{2}{3}\sqrt{\frac{kbd^2N^3}{4.5ps^2}} - (D_1 + D_2 + D_3 \ldots + D_{n-1}) \quad (4)$$

$$D_{n-1} = \frac{2}{3}\sqrt{\frac{kbd^2(N-1)^3}{4.5ps^2}} - (D_1 + D_2 + D_3 \ldots + D_{n-2}). \quad (5)$$

Equation (1) is obtained by substituting equation (5) in equation (4).

The values of $[N^{3/2} - (N - 1)^{3/2}]$ for various values of N are as follows,

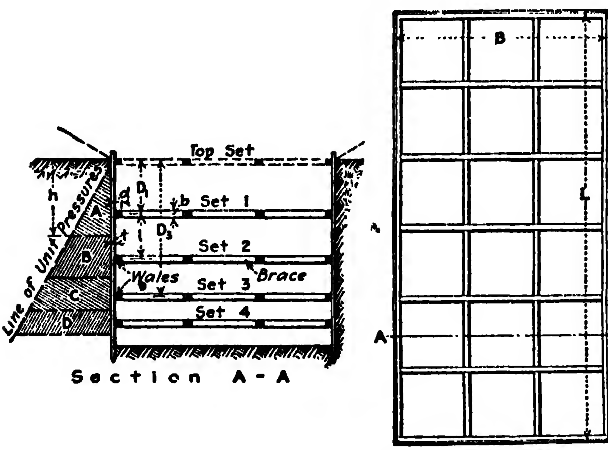


FIG. 79a. Plan, Section and Outside Pressure Diagram of Single-Wall Cofferdam.

N	1	2	3	4	5
$[N^{3/2} - (N - 1)^{3/2}]$	1.00	1.83	2.37	2.80	3.18
N	6	7	8	9	10
$[N^{3/2} - (N - 1)^{3/2}]$	3.52	3.82	4.11	4.37	4.62

Assuming that the sheet piling acts as a simple beam between wales, including the top wale, the highest stressed section will be between set 1 and set 2 and the bending moment may be determined as for the lower section of the sheet pile of Fig. 65*b*.

For hydrostatic conditions and an allowable unit-stress in the timber of 1500 pounds per square inch, equation (1) reduces to

$$D_n = 1.538 \frac{b^{1/2}d}{s}[N^{3/2} - (N - 1)^{3/2}] \tag{6}$$

and with earth-pressure conditions corresponding to a 30-pound equivalent fluid and the same allowable unit-stress in the timber as above it becomes

$$D_n = 2.22 \frac{b^{1/2}d}{s}[N^{3/2} - (N - 1)^{3/2}]. \tag{7}$$

The load on each strut is given by the formula $\frac{kbd^2}{9s}$.

EXAMPLE.—A cofferdam 30 feet deep is to be designed for hydrostatic conditions, with 12- by 12-inch wales, an allowable unit fiber stress in bending of 1500 pounds per square inch and a strut spacing of 8 feet. By equation (6) the values of D_n are 8.0, 14.6, 19.0, 22.4, 25.4 and 28.2 feet.

The maximun moment in the sheet piling is

$$\frac{62.5 \times 11.3 \times 6.6^2 \times 12}{8} = 46,100 \text{ inch-pounds.}$$

If timber piling is used.

$$\frac{1500 \times 12t^2}{6} = 46,100 \; t = 3.92 \text{ inches, hence}$$

4-inch piling will be used. If steel-piling is used, the required section modulus per horizontal foot of wall will be

$$\frac{46,100}{16,000} = 2.88,$$

from which a section may be picked out of a structural handbook.

The load on each strut is

$$\frac{1500 \times 12^3}{9 \times 8} = 36,000 \text{ pounds.}$$

Assuming a minimum diameter of 6 inches, the unit fiber stress allowed by the formula $1500 \left(1 - \frac{1}{60d}\right)$ is 1100 pounds per square inch. Dividing this into 36,000 the required area of strut is found to be 32.7 square inches; therefore a 6- by 6-inch strut will be used

ART. 80. COST OF COFFERDAMS

Little value attends the mere statement of cost of engineering works unless all the conditions are fully described.[1] For this reason only a few figures will be given here. In Art. 73 the cost to the United States of the steel sheet-pile cofferdam at Black Rock Harbor is given.

THOMAS P. ROBERTS writes[2] that for large cofferdams constructed in the rivers near Pittsburgh, Pa., the cost per linear foot of cofferdam will vary from $8 to $10. These cofferdams are of the double-wall type, from 10 to 12 feet wide and from 14 to 16 feet high. Two-inch hemlock sheet-piling is used.

In constructing some piers for a bridge over Paint Creek, near Chillicothe, Ohio, where the water was from 3 to 6 feet deep, a single-wall steel sheet-pile cofferdam, 16 by 62 feet in plan, was used. The piling was 16 feet long and was driven into the gravel bottom until the top of the same was 2 feet above water-level. The bracing consisted of two horizontal wales at the top, running longitudinally and cross-braced with struts. The first cost of the sheet-piling was about $1822, and as the same piling was used for five cofferdams, the cost per cofferdam was about $364. The cost of placing two of the cofferdams averaged $94, while the cost of removing the piling per cofferdam was $47, thus making the total cost of each cofferdam about $505.

In 1886, near the same site, some cofferdams of the double-wall type were built with wooden sheet-piling on guide piles. For the river piers the cofferdams were 22 by 45 feet inside and 35 by 58 feet outside. The guide piles were about 8 feet apart. The wales were 3 inches thick and the sheet-piling was made of 2-inch planking. The bid for the construction of these two cofferdams averaged about $569, the unit prices being as follows: timber, $24 per 1000 feet B. M.; piles, 30 cents per linear foot; iron bolts, 5 cents per pound; and earth filling in cofferdam, 30 cents per cubic yard. At the time the steel cofferdams were

[1] See Engineering News, vol. 70, page 1305, Dec. 25, 1913.

[2] Engineering News, vol. 54, page 138, Aug. 10, 1905.

built (about 1905) the cost of the double-wall cofferdams would probably have been between 30 and 40 percent greater than in 1886. These figures show the considerable economy of cofferdams in which steel instead of timber sheet-piling is employed.

The earth cofferdam illustrated in Fig. 67*c*, built in 1914, cost $19.40 per foot, this cost being made up of the following items: lumber, $4.40; tie rods, bolts, etc., $3.60; handling, framing and placing lumber, $6; and filling and banking with hydraulic dredge, $5.40. The lumber cost $27 per M feet B. M., while the cost of framing and placing was $36.50 per M feet B. M. Tie rods, bolts, etc. cost 3.85 cents per pound threaded. The filling cost 12.2 cents per cubic yard.

ART. 81. CHOICE OF TYPE

The best type to use in any particular case is that one which fulfills all the required functions at a minimum cost. Where the depth of water is not great, and the danger of overflow and washing away does not occur, the simple earth cofferdam will prove the cheapest and most satisfactory, especially if the site of the permanent foundation must be excavated to some depth, for in this case the excavated material may be used to form the cofferdam. Where the depth of water is considerable, the width of the cofferdam becomes so great that this type is not economical.

Where the bottom can be penetrated with piles, the sheet-pile cofferdam with guide piles is a very satisfactory type. For high heads the double-wall cofferdam will be used. This form approximates somewhat the earth cofferdam, but possesses the advantage over it that less earth is required, and it is also a stronger structure and more nearly water-tight. The single-wall type obstructs the waterway less than does that with double walls, but it has less strength. If bracing can be used on the inside, the latter can usually be made sufficiently strong to withstand any forces that are likely to come upon it.

Where the bottom is composed of rock, a sheet-pile cofferdam on a frame or cribs will be used. Frames are used where the

cofferdam can be braced across by struts, but where the structure is too large for such bracing, cribs are necessary. The crib cofferdam may be said to have gone out of use, the open caisson having taken its place. Where the cofferdam is not large and the same size is to be used a number of times, some form of movable structure should be adopted.

Whether wooden or steel piling should be used in any particular case becomes simply a question of the relative cost of the two types. In general, the steel piling will be used in and near cities or where the work is in close proximity to a railroad, while the wooden sheet-piling will be cheaper near centers of timber supplies. Steel piling will also show more economy the greater the depth of cofferdam.

CHAPTER VII

BOX AND OPEN CAISSONS

Art. 82. Definitions and Classification

As defined in the Century Dictionary, a caisson is a "large and water-tight box or casing, in which work is conducted below water-level, as in a bridge pier." Unfortunately, this meaning is true only for a small proportion of the structures now termed caissons. In fact, so many modifications of the original type have developed that further differentiation is necessary. For this reason caissons will, in this book, be divided into three general types: box caissons, open caissons and pneumatic caissons.

All caissons have one characteristic in common: they form a permanent shell for, and are an integral part of, bridge and building foundations, being used simply as a convenient means of placing such a foundation in position.

The box caisson is used where no sinking is required, and consists merely of a box, open at the top and closed at the bottom, which is filled with concrete or other masonry, to serve as a foundation for the pier or other structure to be built on the same. When sinking must be resorted to in order to carry the foundation down to a stratum having sufficient bearing power to carry the superincumbent load, the box must be open at the bottom in order that the earth underneath may be removed to allow sinking to take place. If this excavation is done by dredging through the water, it is called the open caisson, while if the caisson is roofed and air pressure applied to force out the water, to allow workmen to excavate the material by hand, it is called a pneumatic caisson. Briefly then, a caisson is a box; if open at the top and closed at the bottom, it is a box caisson; if open both at the top and at the bottom, it is an open caisson; while if

open at the bottom and closed at the top and it utilizes compressed air, it is a pneumatic caisson. In all cases the caisson is merely a shell which must be filled with concrete or other masonry to form the foundation.

Most caissons are surmounted with cofferdams on account of the undesirability of extending the caisson above low-water level both for durability and for appearance.

ART. 83. BOX CAISSONS OF TIMBER

A box caisson is a box, usually water-tight, closed at the bottom and sides and open at the top, which forms an integral and permanent part of the foundation. Box caissons are made of timber and of concrete, the former material being more widely employed than the latter.

Except where placed on piles, the use of this type of caisson is limited, owing to the necessity of first excavating to the desired depth, *i.e.*, to where firm bearing may be obtained, before placing the caisson. The depth to which it is possible to excavate is limited, owing to the tendency of the wet material to flow into the hole. In a few cases box caissons have been made to sink several feet by running pipes through the bottom and forcing water through the same, thus washing out the material from underneath and allowing sinking to take place.

The box caisson used for the foundation of the Sutherland's River bridge, Nova Scotia, had a bottom composed of a double thickness of 12- by 12-inch timbers laid close, the timbers of the upper course running at right angles to those of the lower course. The sides were formed of vertical studding, horizontal sheathing and diagonal bracing. This caisson, which was built on shore and made water-tight, was launched and towed to the site, after which the permanent masonry was placed to sink it to the bed of piles on which it was to rest.

The pivot-pier caisson of the Newark Turnpike bridge between Jersey City and Kearney Township was circular in plan and was composed of light wooden walls, with an octagonal timber grillage 4 feet thick for the bottom, as shown in Fig. 83*a*.

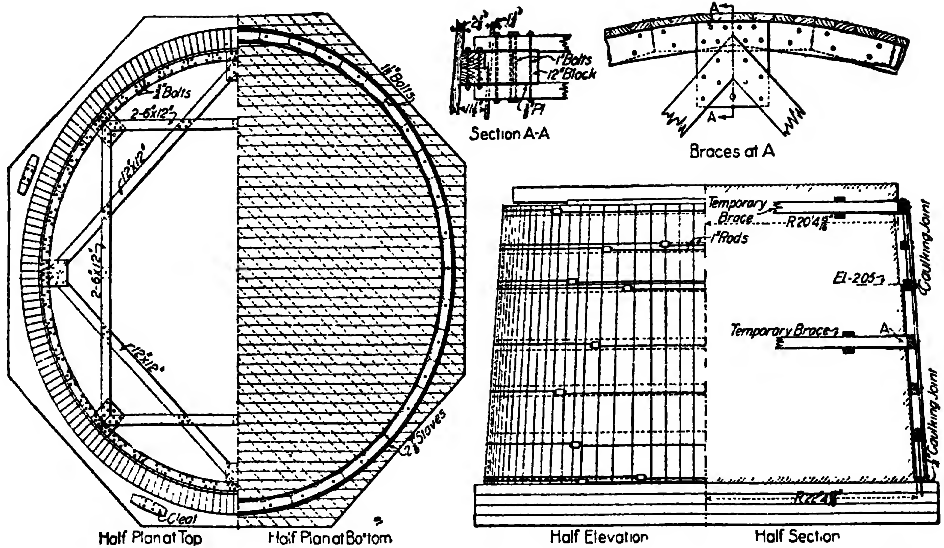


Fig. 83a.—Box Caisson for Pivot Pier of Newark Turnpike Bridge over the Hackensack River.

"After the deposition of 11 feet of concrete in the caisson the horizontal bracing at the fourth ring was removed and a form was built for that portion of the concrete above the top of the permanent caisson. When the concrete reached the top of the cofferdam the bracing at the seventh ring was removed. After the pier was completed the 1-inch bolts in the cofferdam were unscrewed from their fixed lower end nuts and the cofferdam released and removed, together with the upper section of form."

This caisson rested on a pile foundation, and is a good example of a box caisson resembling a movable cofferdam on grillage. The drawing shows that a small amount of interior bracing was used. The caisson, which was 24 feet high, was surmounted above low-water level by a detachable cofferdam 8 feet high, made of the same material as the sides of the caisson.

[1]"The staves, about 32 feet long, were made of 3- by 8-inch timbers, dressed and cut at the mill to the required dimensions. They were tapered $1\frac{1}{16}$ inch, to correspond with the batter of the pier, and had edges beveled to make exterior calking joints. Each stave was made in two pieces to allow for the removal of the cofferdam on top, and the horizontal butt joint between the caisson and cofferdam was calked on the outside. Each stave was bored at the mill with 26 holes for $\frac{3}{8}$-inch spikes to the inside rings made of two courses of timber, breaking joints, bolted together and having their outer edges dressed to circular curves. At eight equidistant points on the upper and middle rings, pairs of horizontal $\frac{3}{8}$-inch connecting plates were bolted to them projecting inward to form jaws which receive the bolted connections of the 12- by 12-inch interior braces . . .

"The first course of 12- by 12-inch grillage timbers was assembled floating in the river and a pair of transverse timbers on top of it were bolted to the outside timbers. These acted as clamps, permitting the timbers to be wedged tightly together and spiked to the second course, after which the clamp timbers were removed and the third and fourth courses laid and spiked tightly together, completing the grillage, which was made of yellow pine and floated with the upper surface about 14 inches above the water. The top-course joints were calked with oakum and a 12-inch coaming was spiked to it, and calked, thus increasing the freeboard and providing a dry working platform when the grillage was submerged deeper by men and materials. The bottom ring, with an outside radius of about $23\frac{1}{2}$ feet, was made of two courses of 3- by 12-inch scarf boards concentric with the pivot, and was spiked to the grillage with $\frac{7}{8}$- by 12-inch dock spikes. Inside of it a light wooden falsework was

[1] Engineering Record, vol. 64, page 720, Dec. 16, 1911.

built to which were secured four concentric horizontal rings, each made with two courses of 6- by 12-inch timber, breaking joints and dressed to circular curves on the outer edges. The lowest of the four rings was anchored to the grillage by 40 2½-inch special lag screws penetrating nearly through the second course of grillage. The lower sections of the staves, 24 feet long, were then assembled to the rings and secured to them by four ⅜- by 7-inch spikes in each stave at each ring."

Caissons of a somewhat different form were used for the foundation of the south pier of the Duluth Ship Canal. They were from 24 to 36 feet wide, 21 feet high and from 50 to 100 feet long. The floor was 8 inches thick laid close, the channel side had a solid wall of a double thickness of 12- by 12-inch timbers, while the opposite side was composed of a single thickness of 12- by 12-inch timbers laid close. Connecting and bracing the two walls were transverse bulkheads of 10- by 12-inch material, spaced 4 feet center to center horizontally.

The caissons were built in the harbor, towed to the site, and sunk by filling with rock and gravel. After sinking, the caissons were covered with a layer of heavy timbers, on which was built the concrete pier, the top of the caisson being slightly below low water-level.

Art. 84. Box Caissons of Concrete

The two principal advantages possessed by the concrete box caisson over the timber caisson are: First, the caisson may be carried up to above low-water level, thus eliminating cofferdam work; and, second, it is a more durable type, especially in those waters infested by marine wood borers. Caissons made of concrete will usually prove somewhat more expensive than those made of timber.

Figure 84*a*[1] shows the type of caisson used as breakwaters and piers for a bridge forming a yacht landing at Glen Cove, Long Island. To make the launching easier, the caissons were built

[1] From paper on Reinforced-Concrete Pier Construction by Eugene Klapp, Transactions of the American Society of Civil Engineers, vol. 70, page 448, December, 1910.

in two sections. They were reinforced for exterior pressures which the structures would meet in sinking and for interior pressures which would occur at low tide by reason of the interior filling.

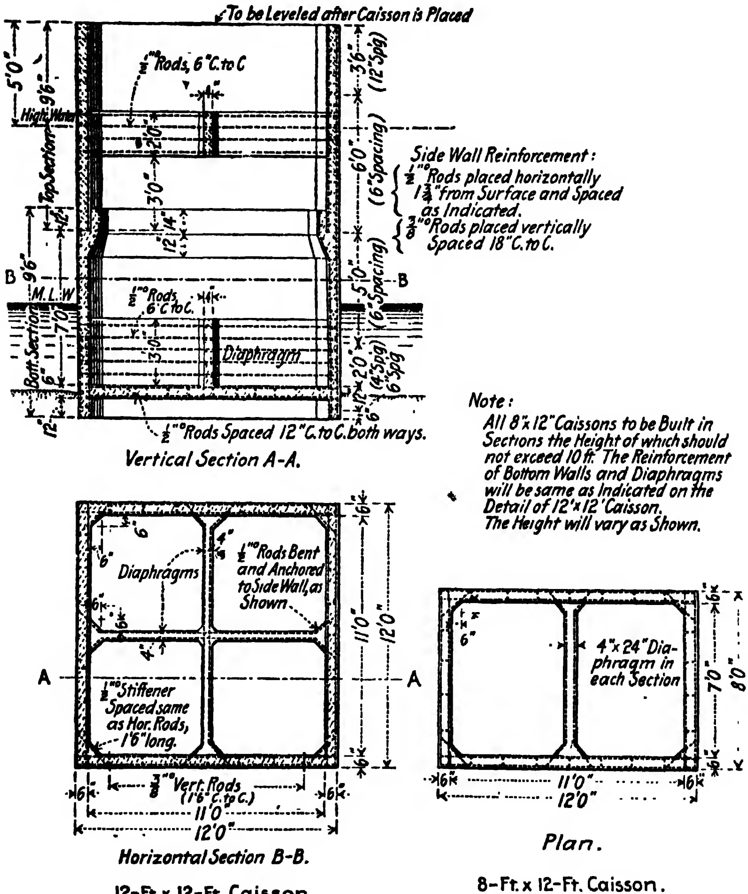


FIG. 84a.—Box Caisson of Reinforced Concrete near Glen Cove, Long Island.

All caissons were cast standing on skids. When the concrete hardened sufficiently, a derrick scow lifted and set them in the water, after which they were towed to position and sunk. Some of the upper sections were placed on the scow and lifted directly

from there upon the lower sections already in place. In the bottom sections a 3-inch hole was cast, which was closed while the caisson was being towed to position. When directly over the site of the foundation bed, water was let in by unplugging the hole to sink the structure, after which the same was filled with sand and gravel.

For a very complete discussion of the subject of concrete caisson construction for breakwaters the reader is referred to a paper by Major W. V. JUDSON in the Proceedings of the Western Society of Engineers, vol. 14, page 533. An abstract of this paper may be consulted in Engineering News, vol. 62, page 34, July 8, 1909.

ART. 85. MISCELLANEOUS TYPES

A metal box caisson, composed of a vertical cylinder 9 feet in diameter and nearly 13 feet high, having sides formed of ½-inch steel plate and a bottom composed of a ribbed and flanged casting, slightly convex downward, was sunk about 13 feet through quicksand to form the foundation for some machinery at the General Electric Company's plant at Schenectady, N. Y. Through the bottom there were 44 1½-inch holes, each hole being tapped for a vertical pipe, which in turn was connected to a 3-inch main which provided water at 80 pounds pressure. Valves were so placed that any combination of streams could be used at once. The caisson was sunk by opening the valves, thus forcing the water through the pipes to scour out under the bottom of the cylinder. At the same time the caisson was heavily weighted with pig lead. Some little trouble was experienced in keeping the caisson plumb but by using certain jets at certain times and by placing the loading material mostly on the high side the structure was finally brought to bearing on the firm material. The pipes were then disconnected near the bottom and concrete placed in the caisson.

Another modified form of box caisson used at the same place to avoid danger of undermining adjacent structures resting on the quicksand was sunk by boring out the quicksand from under the caisson. A pile foundation was to be used and as it was

desired to cut off the piles at an elevation well below ground water-level, a box caisson 4 feet high and 16 by 40 feet in plan, inside dimensions, was constructed with a large number of 12-inch holes cut through the bottom. Twelve-inch vertical wrought-iron pipes, 4 feet long and open at both ends, were fitted into these holes, after which the caisson was filled with concrete to the top of the pipes. After this had set, 3600 tons of pig iron were loaded on the caisson between the pipes. The quicksand would not rise in the pipes, but by means of post-hole augers the material was raised and removed. Care was taken to remove but a slight amount from a hole at any one time in order to prevent unequal settlement. After sinking the desired amount, about 4 feet, piles of about 11 inches diameter at the top were driven through the pipes to a distance of about 19 feet below the bottom of the caisson and were then cut off level with the tops of the pipes, after which the latter were grouted and the foundation completed.

In both the foregoing cases the problem was to sink a foundation through quicksand without disturbing adjacent structures founded on quicksand. Success was due to weighting the caisson so heavily that quicksand could not flow under the same from outside and at the same time providing means to remove the sand under the caisson.

ART. 86. SINGLE-WALL OPEN CAISSONS

An open caisson is a box-like self-contained structure either partly or entirely open at both top and bottom, and forming an integral and permanent part of the pier.

The open caisson which forms one of the most important classes of structures used for subaqueous work, and has the distinction of being employed for the deepest foundations, may be divided into three types: first, the single-wall timber caisson, consisting of a frame with solid walls and without top, bottom, interior chambers, or cutting edges; second, the cylinder caisson, consisting of open cylinders of iron or masonry; and, third, the caisson with dredging wells, consisting of a struc-

ture partly closed both at the top and at the bottom, with open wells running vertically through it. The first type is used where little or no sinking is required, while the second and third are employed where sinking is necessary; the second where the required cross-sections of foundations are small and the third where they are large.

TIMBER.—In the details of construction the single-wall open caisson of timber resembles the single-wall crib cofferdam (Art. 74) and differs from it chiefly in that it is an integral part of the foundation. The caisson usually consists of a solid framework of 12- by 12-inch timbers thoroughly calked. It is used only where little or no sinking is required or else where the material to be sunk through is very soft, because sinking must be done by artificially weighting the structure with removable material, such as iron rails. If soft material covers the site, as much of it as possible should be dredged out before placing the caisson. Removing the material from within the caisson after it is placed, and also using the water-jet along the sides, will greatly facilitate sinking. On reaching its final position concrete is deposited through the water to a depth of several feet and allowed to harden. This virtually forms a box caisson which is then pumped out and filled with concrete, placed in the dry, to make the foundation for the pier. It is customary to add a cofferdam on top of the caisson so that the latter may not extend above low water.

Figure 86*a* shows the details of the caisson used for one of the piers of the French River bridge of the Canadian Pacific Railway. The lower part was built on shore, the structure then being launched and completed in the river. Rolls of canvas were attached to the inner faces along the bottom and as soon as the caisson was lowered to the bottom divers went down and spread out the canvas, and on it laid bags of cement to close the openings under the walls. A layer of mortar was then deposited through the water on the rock bottom, after which the remainder of the caisson was filled with concrete. The caisson was surmounted with a cofferdam of exactly the same construction as the caisson.

Figure 86*b* shows the details of the caissons used in the substructure of the Columbia River bridge of the Oregon-Washington Railroad & Navigation Company. The river bed was

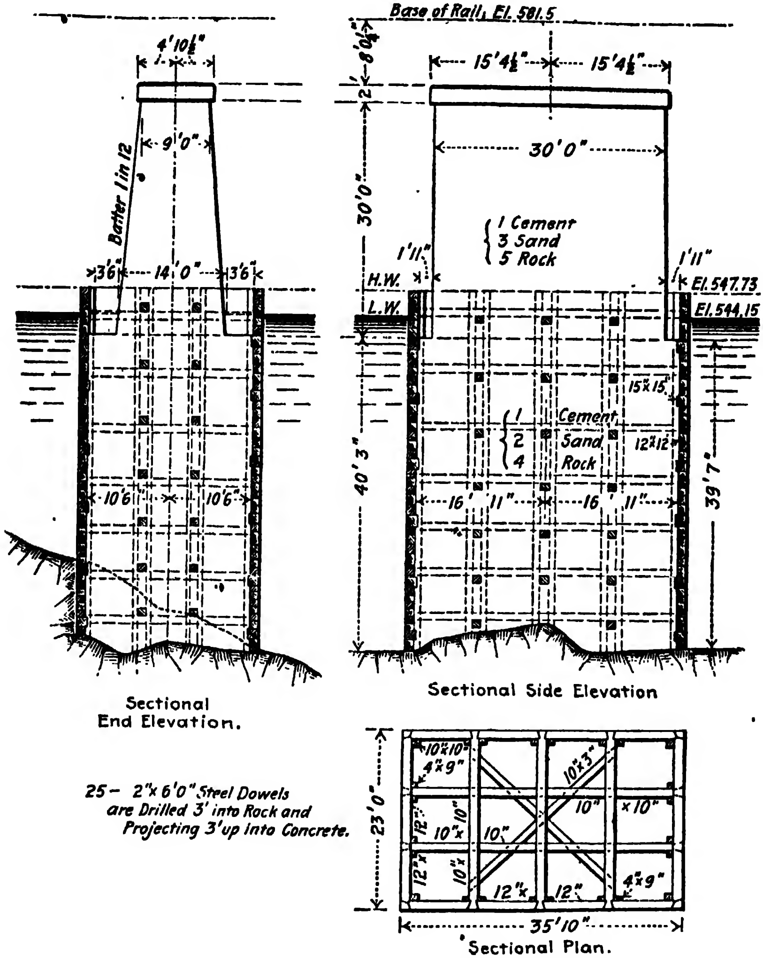


FIG. 86*a*.—Open Caisson for Canadian Pacific Railway Bridge over French River.

composed of very firm soapstone, overlaid in places with cemented boulders, gravel and sand. The maximum depth of water at the usual stage of the river was about 30 feet, with a maximum velocity of current of 7 miles per hour.

The caissons after being framed were floated to place and sunk by loading with steel rails, the latter being placed in the racks shown on the drawings. All concrete was placed through the water, no attempt being made to pump out the caisson. Upon completion of concreting, all timbers above low water-level were removed.

The piers of the Interstate bridge over the Columbia River at Portland were founded on caissons, 16 by 57 feet, resting on

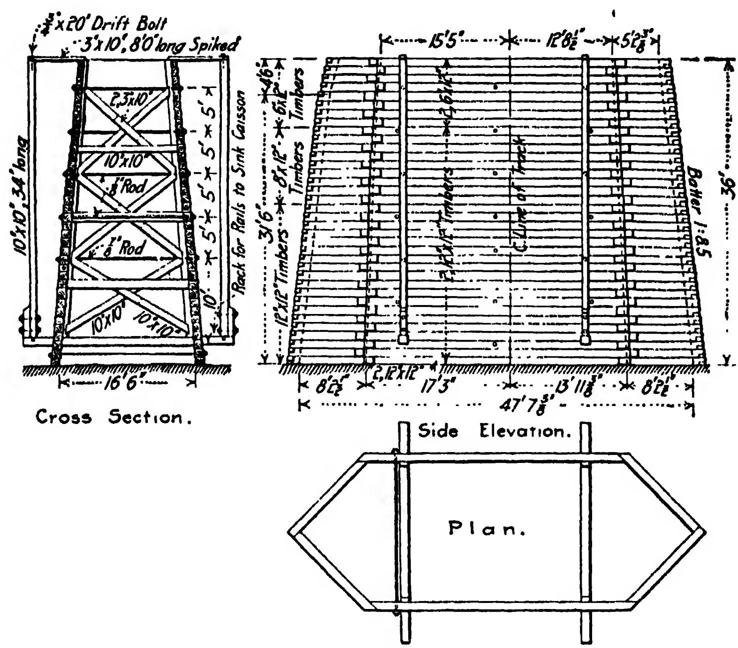


FIG. 86b.—Open Caisson for Piers of Oregon-Washington Railroad and Navigation Company over Columbia River.

pile foundations, as shown in Fig. 86c. The bed of the river consists of sand, extending to a great depth, which is subject to considerable scouring action. The cribs were sunk from 20 to 25 feet below the river bed and the piles were jetted down inside the crib after the material had been excavated. Concrete was then deposited through the water and after this had hardened the water was pumped out, the piles cut off a short distance

below low water and the remainder of the concrete placed in the open.

The concrete placed under water was deposited through a tremie consisting of a 10-inch wrought-iron pipe with a 2-cubic

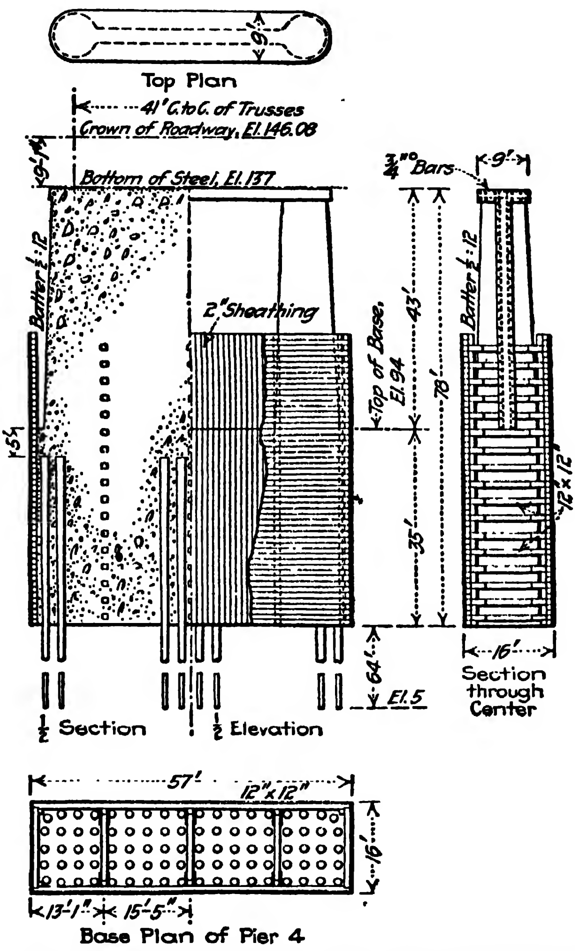


FIG. 86c.—Timber Caisson of Interstate Bridge across Columbia River, Portland, Ore.

yard hopper at the top. After the first charge of concrete was placed, the lower end of the tremie was constantly immersed in the soft concrete, the full length of the pipe being kept filled

with concrete. On filling the hopper, the tremie was lifted slightly until the pressure was sufficient to force the concrete

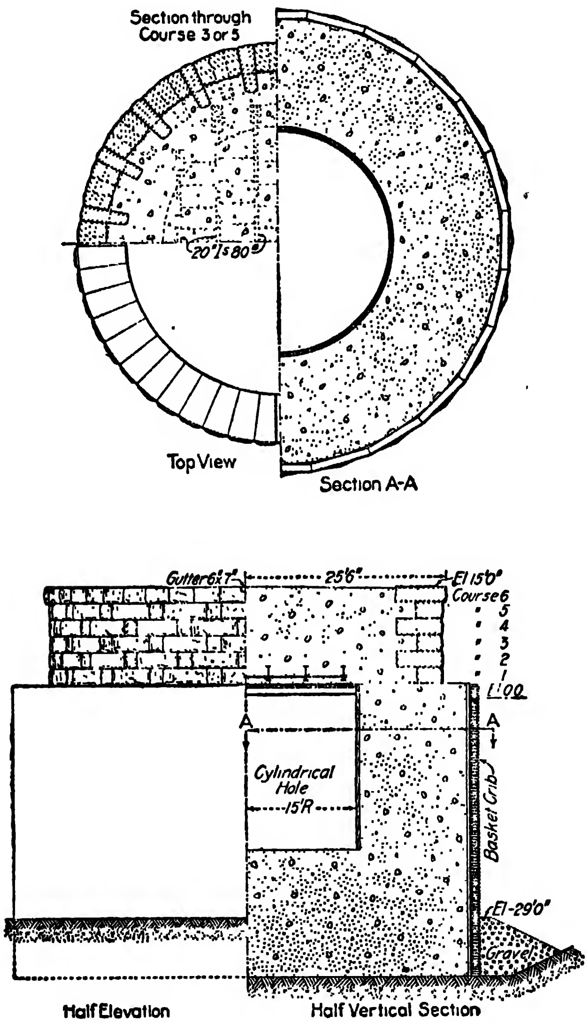


FIG. 86d.—Basket-Crib Type of Open Caisson.

out and when the hopper was nearly empty the tremie was lowered again.

The piles were so embedded in the caisson that they formed an integral part of it, so that the pier would still be stable even though some unusual scour extended below the bottom of the base.

To sink the caissons of the Rio Conchos bridge of the Kansas City, Mexico and Orient Railway, stringers were placed across the ends and a floor was placed on the portions of these stringers projecting beyond the sides, and these floors were boxed up to a height sufficient to carry a load of 75 tons of gravel each. This gravel was dredged from the inside of the caissons.

Figure 86*d* illustrates an example of the circular caisson of a form called the basket crib. This type of caisson has been used by the Engineering Department of Boston, Mass., in a number of cases for the foundations of pivot piers. This caisson, which was 60 feet in diameter and 40 feet high, is of special interest on account of the cylindrical chamber, 30 feet in diameter and 22 feet high, in the upper part of the caisson, which reduced the volume of concrete. This plan follows the tendency of the present day in regard to bridge piers (Chap. XII). [1]"The basket crib, or form for the pier foundation, was built of about 145 horizontal courses of 3- by 12-inch yellow pine planks, 8 feet long, laid flat and breaking joints. The ends were beveled to make radial joints, and each plank was secured to those below it by 1-inch oak tree nails 9 inches long, two at each end of each plank . . . In addition, the planks were well spiked to the lower courses throughout their entire length with 6-inch spikes. The courses were also secured together by 4- by 12-inch vertical planks opposite alternate joints which were fastened to the inner circles of the crib by lag screws."

Before placing the caisson the site was dredged to rock. The caisson was sunk by loading with old iron and stone and by hanging heavy chains over the walls.

CONCRETE.—Reinforced-concrete caissons were used in the construction of a bridge over the Platte River near Bridgeport, Neb., as illustrated in Fig. 86*e*. Owing to the shallow water,

[1] Engineering Record, vol. 68, page 138, Aug. 2, 1913.

the caissons were built in place, with 12- by 12-inch timbers for the cutting edge, two to five courses being used, depending the depth of the water. The boxes were sunk by excavating with a clamshell bucket and by loading with concrete piles, as much as 160 tons being used in some cases.

After a caisson had reached its proper depth, reinforced-concrete piles were sunk by jetting, their tops being about 5 feet below the top of the caisson. After all the sand in the

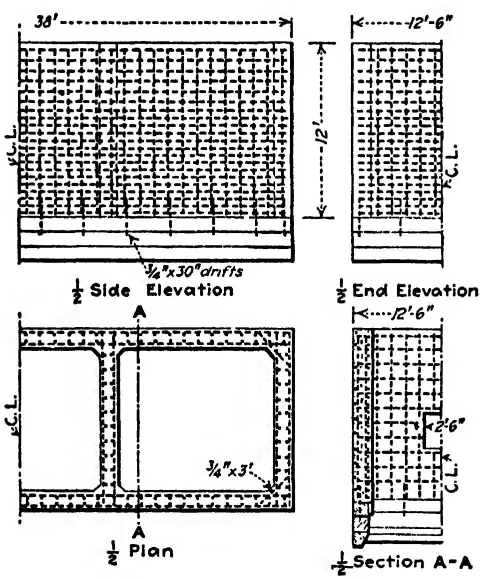


FIG. 86c.—Open Caisson of Concrete.

caisson around the piles had been removed by an hydraulic sand ejector, a 4-foot sealing course was placed by means of a tremie. After this concrete had set for 24 hours, the box was unwatered and the rest of the concrete placed.

ART. 87. CYLINDER CAISSONS

The cylinder caisson consists of a cylindrical shell of masonry, wood, iron or reinforced concrete, shod with some form of cutting edge, and is sunk by excavating the material within the caisson and at the same time weighting it, or using the water-jet

around the sides to decrease the friction. Where the cylinder is of large diameter there may be two shells, an outer and an inner one, the space between the two being filled with concrete as the caisson sinks. Where the cylinder caisson is used it is customary to construct the piers as an upward extension of the caisson.

This type of foundation is widely employed where the loads to be supported are not great but where it is necessary to go down a considerable distance to avoid scouring action. Particularly in the British provinces of the Far East has it been widely used, for there the rivers are dry, or nearly so, for a large part of the year, but deep and torrential during certain months, thus requiring the foundations to be bedded at a depth below that of any possible scour.

CYLINDER CAISSONS OF MASONRY.—For many centuries the natives of East India have employed the masonry caisson, or "open well," as it is more frequently called, in sinking the foundations for their bridges. In their most primitive form these caissons consisted of wells large enough for but one man to work in—about 3 feet in diameter—and were built of brick masonry resting on wooden curbs. They were sunk to a maximum depth of about 17 feet by divers excavating inside of them and bringing up the excavated material in buckets. When bedded on a firm stratum they were filled with masonry. For those streams which were low or dry for much of the year this was a cheap and effective method of placing the foundations.

A modern example of this general type is found in the construction of the north abutment caissons of the Chittravati bridge, where brick caissons on wrought-iron curbs were used, the exterior diameter being 12 feet and the thickness of the brick wall 2 feet. They were sunk to a maximum depth of about 63 feet by dredging through the wells and by loading with iron rails.

CYLINDER CAISSONS OF WOOD.—Figure 87*a* illustrates the caissons of wood used for the foundations of a shipping pier at San Francisco. Each caisson was made of Douglas fir timber staves 4 inches thick, banded with iron. The bottom consisted

of a cast-iron bell attached to the staves as shown in the diagram. The cylinders were bedded about 12 inches in a hard clay stratum which was about 40 feet below mean low water and 46 feet below mean high water. An average depth of about 15 feet of soft mud overlaid the clay.

The caissons were sunk through this soft material by means of four water-jets playing around the bottom of each. Little driving was necessary until the clay stratum was reached.

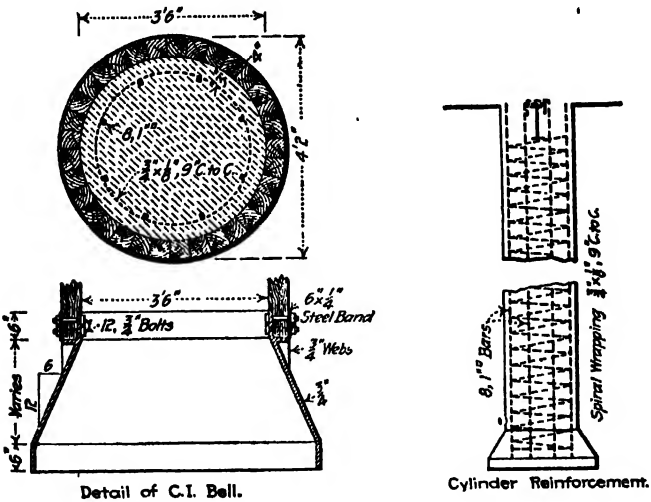


FIG. 87*a*.—Cylinder Caisson for Foundations of Shipping Pier at San Francisco, Cal.

Special frames were used on top of the caissons to receive the blows of the hammer. As soon as the desired penetration was obtained, the water and mud were pumped from the caisson—the clay effectually sealing the bottom—and the bottom carefully inspected, after which the reinforcement was placed and the cylinder filled with 1-2-4 concrete. This type of caisson has been used but little on account of its lack of strength and rather high cost.

ART. 88. METAL CYLINDER CAISSONS

Experience has shown that there are many advantages gained by using a shell of iron or steel in place of one of brick masonry,

especially when water covers the site of sinking. The shell may be of cast iron, wrought iron or steel, the last being used almost exclusively in this country. The metal type possesses three advantages over the masonry: first, greater strength; second, a higher degree of water-tightness, and, third, less friction developed in sinking. After the caisson is sunk to a proper bearing it is filled with concrete or sand, the former being invariably used in America, while English engineers use the latter to a considerable extent.

The California City Point coal pier offers a good example of the use of cylinder caissons of small diameter, being formed of flanged cast-iron pipe 4 feet in diameter. The depth of water at the site was about 30 feet, while from 4 to 40 feet of mud overlaid the hard bottom on which the caissons were to rest.

In order to increase the bearing area on the bottom a special conical section was made for the lower end of the cylinders, the maximum diameter of this section being 8 feet. The shell above this lower section was composed of regular 4-foot cast-iron pipe in 12-foot lengths, each section being fastened to the one above and below with 44 1⅜-inch bolts through the flanges.

Sinking the shells was accomplished as follows: A number of sections were bolted together, lowered vertically to position on the mud bottom and braced there with guys. They were sunk by dredging out the inside of the pipe by means of a ½-cubic-foot orange-peel bucket, new sections of pipe being added as the cylinder went down. Where the resistance to sinking was considerable, the work was facilitated by temporarily loading the top of the caisson with steel beams and girders, and by the use of the water-jet around the cutting edge. After sinking operations were completed, the conical portion of the cylinder was filled with concrete deposited through the water, and after the latter had hardened the water was pumped from the cylinder. Fourteen vertical reinforcing rods of 1$\frac{3}{16}$-inch diameter were then placed in each cylinder, after which the latter was filled with a 1-3-5 mixture of concrete, rammed in 12-inch layers, to form the foundation for one of the columns of the coal pier.

The caissons for the highway bridge across the Kansas River at Fort Riley are typical examples of steel cylinder caissons, so much used for light highway bridges, where it is necessary to go down some distance below the bed of the stream to get proper bearing material. Each pier consisted of two cylinders well braced together, each cylinder being 5 feet in diameter. The metal used was ¼ inch thick, although a thickness of ⅜ inch would have been better. The cylinder sections were in 6-foot lengths, butt-jointed with splice plates on the inside, and were riveted up in 12- and 18-foot sections in the bridge shop. The cylinders were 54 feet long and were sunk through fine sand by dredging and weighting, and at a time when the river was dry, to an average depth of 24 feet below the river bottom.

The 8-foot diameter cylinders used on the Atchafalaya bridge, Morgan City, La., are said to be the deepest single-wall cylinder caissons on record, being sunk to a depth of 120 feet below high water, or from 70 to 115 feet below the mud line. Below this mud line the cylinders were of cast iron and above of wrought iron (see BAKER'S Masonry Construction).

English engineers have used cast-iron cylinder caissons of fairly large diameter for many of their bridges, but experience (Charing Cross bridge, 1860) long ago taught them that the lower sections should be of wrought iron or steel, for cast iron is too brittle to use for a cutting edge. For a very complete description and discussion of the sinking of cast-iron cylinder caissons with wrought-iron cutting edges, see Proceedings of the Institution of Civil Engineers, vol. 103, page 135.

One of the most expensive items connected with the sinking of cylinder caissons is that of artificially weighting the structure to promote sinking, because this weighting material must be removed and replaced each time new sections are added to the caisson. Largely for this reason, where the size of the caisson will permit, it is advisable to use a double wall so that much of the permanent concrete filling may be placed during sinking, and thus decrease the amount of temporary loading necessary.

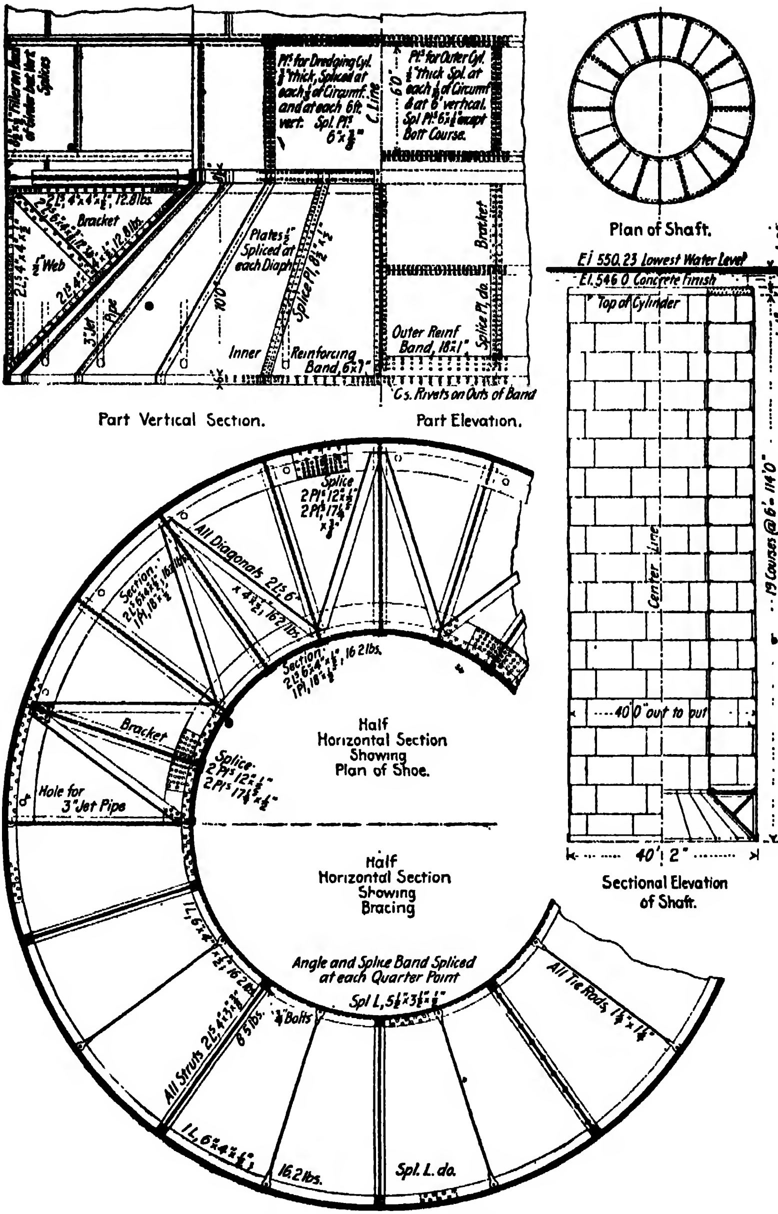


FIG. 88a.—Details of Open Caisson for Pivot Pier of Omaha Interstate Bridge. Designed in 1902.

This was done in the caisson for the pivot pier for the Omaha Bridge and Terminal Company's bridge across the Missouri River from East Omaha to Council Bluffs, Iowa. As shown in Fig. 88*a*, the caisson, which was of steel, had an outer diameter of 40 feet and an inner diameter of 20 feet. It rested on solid rock 120 feet below low water. For the first 50 feet the material was sand and clay, and below this there was about 60 feet of coarse sand overlying a few feet of boulders which rested

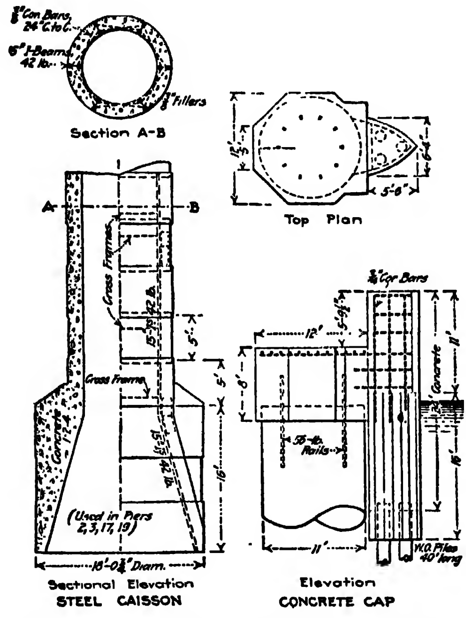


FIG. 88*b*.—Double-Wall Metal Cylinder Caisson.

on solid rock. At a low stage of the river the depth of water was slight. Sinking was accomplished by a combination of three agencies: dredging the material from inside the caisson, using water-jets to reduce the side friction and filling the space between the two shells with concrete. On completing the sinking, the well was also filled with concrete. The details of the caisson are clearly shown in the illustrations.

Figure 88*b* shows the type of caisson used by the Chicago, Burlington, & Quincy Railroad on its bridge across the Platte

River near Ashland, Neb. The caisson was lined with 18 inches of concrete, the diameter of the well being 8 feet. For the most part, 3/8-inch metal was used, although in some cases the first four sections just above the flaring part were of 3/4-inch steel.

ART. 89. REINFORCED-CONCRETE CYLINDER CAISSONS

The use of reinforced concrete for this type of caisson is likely to increase greatly in the future, since it is the most appropriate material, as explained in Art. 93.

One of the early examples in America of the application of this type of caisson was placed in 1910 in the foundations of the pedestals for the Penhorn Creek viaduct of the Erie Railroad. A single caisson was used for each pedestal. The shell of the caisson consisted of a hollow reinforced-concrete cylinder, having an exterior diameter of 6½ feet and an interior diameter of 4½ feet, thus giving a thickness of 1 foot. It was reinforced with ½-inch vertical rods, spaced 9 inches center to center, and by ½-inch horizontal circular rods spaced 6 inches, the former located 2 inches from the outside face and the latter just inside of these. The depths to which the caissons were sunk varied greatly, many of them extending to about 70 feet below the surface of the ground, which corresponds to about 55 feet below ground-water level.

In constructing a caisson, a pit about 11 feet square and 10 feet deep was excavated and lined with 3- by 12-inch butt-jointed sheathing, braced by 12- by 12-inch horizontal rangers. Four vertical 12- by 12-inch sticks were then placed, one at the middle of each waling piece, to serve as a guide for the caisson. The cast-iron cutting edge, shown by the heavy lines in Fig. 89*a*, was then placed in the bottom of the pit. Above this were placed outside and inside collapsible steel forms in 5-foot lengths. All caissons were cast in 20-foot units, the caisson being built to this height, allowed to set, sunk and another section added, the whole operation being repeated until the desired depth was reached. Each section was allowed to harden six days before it was sunk.

Sinking through the mud and sand was effected for the most part by interior excavation with an orange-peel bucket. The water-jet was used to some extent and weighting was also resorted to at times. It was found advantageous to keep the jet pipes separate from the caisson and to work them by hand. The average rate of sinking through mud was 6½ feet per day,

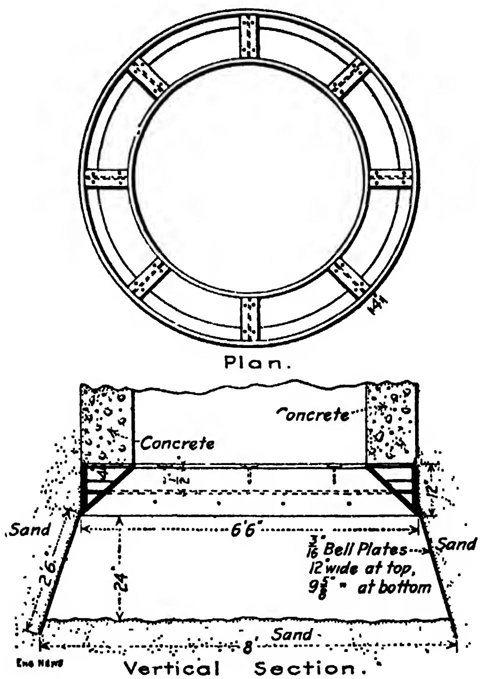


FIG. 89a.—Cutting Edge of Caisson, Penhorn Creek Viaduct, Jersey City, N. J.

while through the dense underlying sand only about 1½ feet per day could be accomplished.

It was at first intended to found the caissons on rock, using an allowable bearing pressure of 10.8 tons per square foot, but later, owing to the greater depth of the rock, it was decided to found them on the dense sand above, which it was thought would safely bear a load of 7 tons per square foot. In order to reduce the unit pressure to this amount, the bottom was belled

out as shown in the illustration. The conical or belled section, which consisted of a number of 3/16-inch steel plates, was placed by a diver who, with the aid of a water-jet, forced the dense sand from around the cutting edge and placed the plates. Each plate was forced into the sand a slight distance at the bottom and sprung behind the cutting edge at the top. Upon the completion of this work the caisson was filled with 1–2½–5 concrete.

Figure 89*b* shows details of the caisson used by the Chicago, Burlington, & Quincy Railroad in the construction of a bridge over the Platte River near Ashland, Neb. These caissons were sunk about 50 feet. One disadvantage of this type of construction is the length of time involved in waiting for the concrete to harden after each section is poured. To eliminate this feature, cylinders may be made of pre-cast sections, with suitable devices for connecting the reinforcement of the various sections.[1]

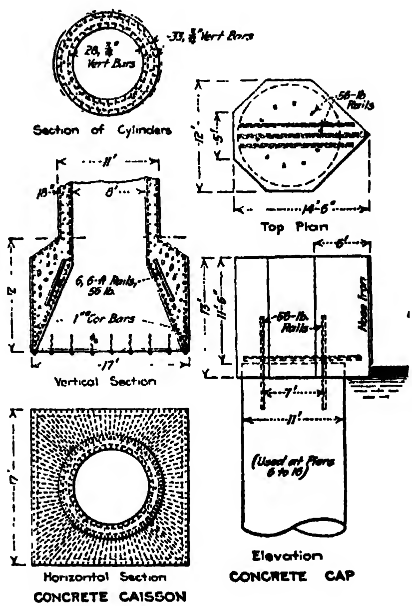


FIG. 89*b*.– Reinforced Concrete Cylinder Caisson.

Pre-cast concrete pieces were used in some construction work in the Far East. For each cylinder there was a cutting-edge ring 3½ feet high, with 11½-foot outside and 9⅓- foot inside diameters. On top of this ring was placed a ring tapering from 11½ to 8 feet outside and from 9⅓ to 5 feet inside diameter. This diameter was maintained to the top, the sections being 5 feet high. Sinking was effected by inside clam-shell excavation and by loading with cast-iron billets.

[1] A good example of this may be found in the Engineering Record, vol. 66, page 60, July 20, 1912.

In contrast to the thick walls just noted, in the construction of a bank building in Stockholm, the walls were only 5 inches thick, but heavily reinforced. Each section had an inside diameter of 7 feet and a height of 2⅔ feet. The sections were provided with recesses and with vertical bars for fastening the sections together. Sinking was accomplished by pumping out the sand and gravel with a centrifugal pump and suction hose, the material at the bottom being loosened by means of a water jet handled by a diver. The excavated material was delivered from the centrifugal pump to a settling basin, from which the water was drained off over a weir and conducted back to the well.

ART. 90. OPEN CAISSONS WITH DREDGING WELLS

This type is based on the same principle as the double-wall open-cylinder caisson, differing from the latter only in the matter of shape and size, although some open-cylinder caissons are as large as those to be classified under the head of open caissons with dredging wells. Perhaps the most notable difference lies in the fact that the open-cylinder caisson has but one dredging well, while the type to be treated in this and the three following articles always has more than one.

The open caisson with dredging wells is a type of construction which has been employed for the deepest foundations ever used for bridge piers. Theoretically, there is no limit to the depth to which this class of caissons may be sunk. The essential principle of the construction is a box-like structure of wood, iron or reinforced concrete, with ballast pockets in the same and with open wells running vertically through it, these wells flaring out at the bottom to practically the whole area of cross-section of the caisson. Through the wells, by means of dredges, the material is excavated from the bottom, and this, together with simultaneously filling the pockets with concrete, is usually sufficient to sink the structure. As in the other forms of caissons, when the structure is sunk to good bearing material the wells are filled with concrete.

The great advantage of this type of structure for foundations is that all the work is done above water, so that the cheapest class of labor can be employed, thus under favorable conditions making a very economical foundation. In the use of the open-caisson method there are three disadvantages, which are absent in either the cofferdam or the pneumatic-caisson process. They are as follows: First, the character of the bottom on which the caisson finally rests can never be as satisfactorily known as when it is possible to exclude the water and inspect the bottom in the dry, nor can the latter be leveled and cleaned as easily as when the other methods are employed; second, the concrete which is placed in the bottom of the well must be placed through the water and consequently is not as good concrete as when placed in the dry; and, third, it is difficult to estimate the possible rate of sinking owing to the trouble which boulders and sunken logs will offer when encountered under the cutting edge. In spite of these disadvantages, the open caisson with dredging wells is widely employed for depths greater than can be satisfactorily handled by the cofferdam process and where the cost of the pneumatic-caisson process prohibits its use, or where the depth is greater than 110 feet below water-level, the limiting depth in practice for pneumatic-caisson work.

Since the sinking of caissons with dredging wells is not under thorough control, the caissons should always be made large enough to allow a moderate amount of deviation from the correct position. As the wells or dredging tubes are the chief means by which the descent can be regulated as regards direction, these should be so placed as to facilitate this object. For instance, when one end of the caisson strikes soft material and consequently sinks faster than the other end, the caisson can be brought back to a vertical position by dredging solely from the high-end well; but, on the other hand, if the wells are distributed along the longitudinal center line, as was the case in the Hawkesbury bridge caissons (Art. 92), and Fraser River bridge caissons (Art. 91), and one side strikes softer material than the other side, it becomes difficult to keep the structure from tilting. A

better arrangement, consisting of two longitudinal rows of wells, was used in the Willamette River bridge caissons (Art. 91).

ART. 91. CONSTRUCTION WITH TIMBER

In America the pneumatic form of caisson, although more expensive, has been preferred to the open caisson; but for the relatively small number of the latter that have been used, because of the low cost of timber, the wooden type has been employed in most cases. In addition to the advantage of low cost, the wooden type is easily built and makes a strong and elastic caisson.

Figure 91*a* shows the details of the deepest caisson, that for Pier 4, of the Fraser River bridge at New Westminster, B. C. This caisson was of timber, and was sunk to a depth of 135 feet below water-level. The outside walls were built of solid courses of 12- by 12-inch timber, sheathed on the outside with vertical 2-inch planks. These walls were built on a solid triangular-shaped timber base of 12 courses of material, the inside of this base also being sheathed with 2-inch planks. The dredging wells were framed with both longitudinal and transverse timbers, laid solid near the bottom but open above this, and sheathed with 2-inch material. The outer- and well-wall timbers formed a series of pockets which were filled with concrete during the sinking of the caisson. All the seams, both in the horizontal timbers and vertical sheathing, and in the upper two courses of the solid timber base, were thoroughly calked.

The caisson was built to a height of about 14 feet on ways on the shore, and then launched and towed to the site where it was to be sunk. Here an 8-inch layer of 1–2–3 concrete was placed on the deck and allowed to harden for a few days to increase the water-tightness of the pockets. The caisson was then gradually built up, concrete being added simultaneously with the building, thus causing the structure to sink. At the start the depth of water was about 50 feet but before the sinking was completed this depth had increased to 65 feet, due to scour. The caisson was guided in sinking by means of long piles. As

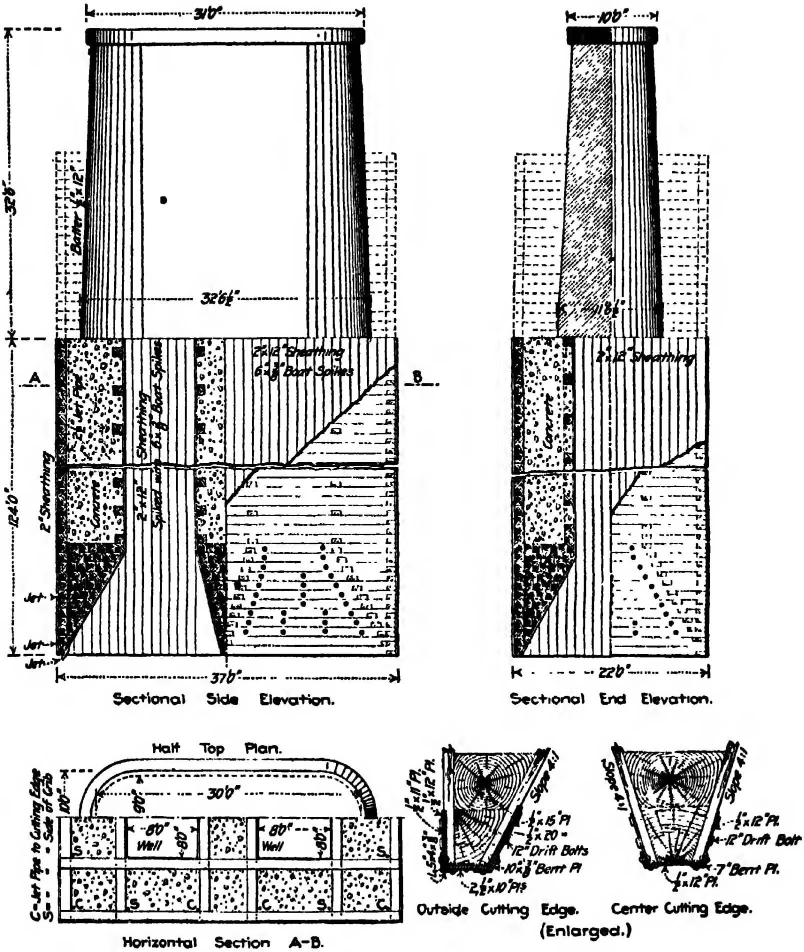


FIG. 91a.—Open Caisson for Pier IV, Fraser River Bridge, New Westminster, B. C.

soon as the river bottom was penetrated a few feet, the concrete in the pockets was built up above water-level and all but the 2-inch sheathing was omitted around the wells. The material penetrated was mostly sand and silt, and 85 percent of this was removed by the sand- and mud-pump process (Art. 94). The caisson finally rested on a bed of compact gravel 135 feet below the normal stage of the river. In sinking, the water-jet was used, the jet pipes being in the positions shown in the illustration.

On completion of the sinking, concrete was deposited in the chamber formed by the flaring out of the wells and this was followed by filling the wells. This concrete, to a depth of 70 ft., was deposited through the water and the remainder was placed in the dry, the water being pumped out of the wells previous to placing the latter.

The caisson for pier 3 of this bridge, when well down in the sand, encountered sunken logs, causing it to tilt. One of these logs had a diameter of 2 feet and extended clear across the caisson. This was removed by boring holes in it with an auger 100 feet long, the point being set by a diver. In these holes were placed charges of dynamite, which on exploding blew the log to pieces. The top of the caisson was surmounted with a cofferdam, as the elevation of the top was about 10 feet below ordinary water-level.

Figures 91*b* and *c* show the details of construction for the 36- by 72-foot open caisson of the Williamette River bridge, of the Oregon-Washington Railroad and Navigation Company. There were six wells, each 9 by 10 feet in the clear, flaring out at the bottom to occupy the whole area of the caisson. The walls of the wells and the outside walls of the caisson were made of a single thickness of 12- by 12-inch timbers laid close, the latter being sheathed on the outside with 3-inch material. As shown in the plan, certain timbers of the well walls were extended the entire length and breadth of the caisson to brace the same.

The lower part of the caisson consisted of V-shaped walls and bulkheads, there being two of the latter running transversely

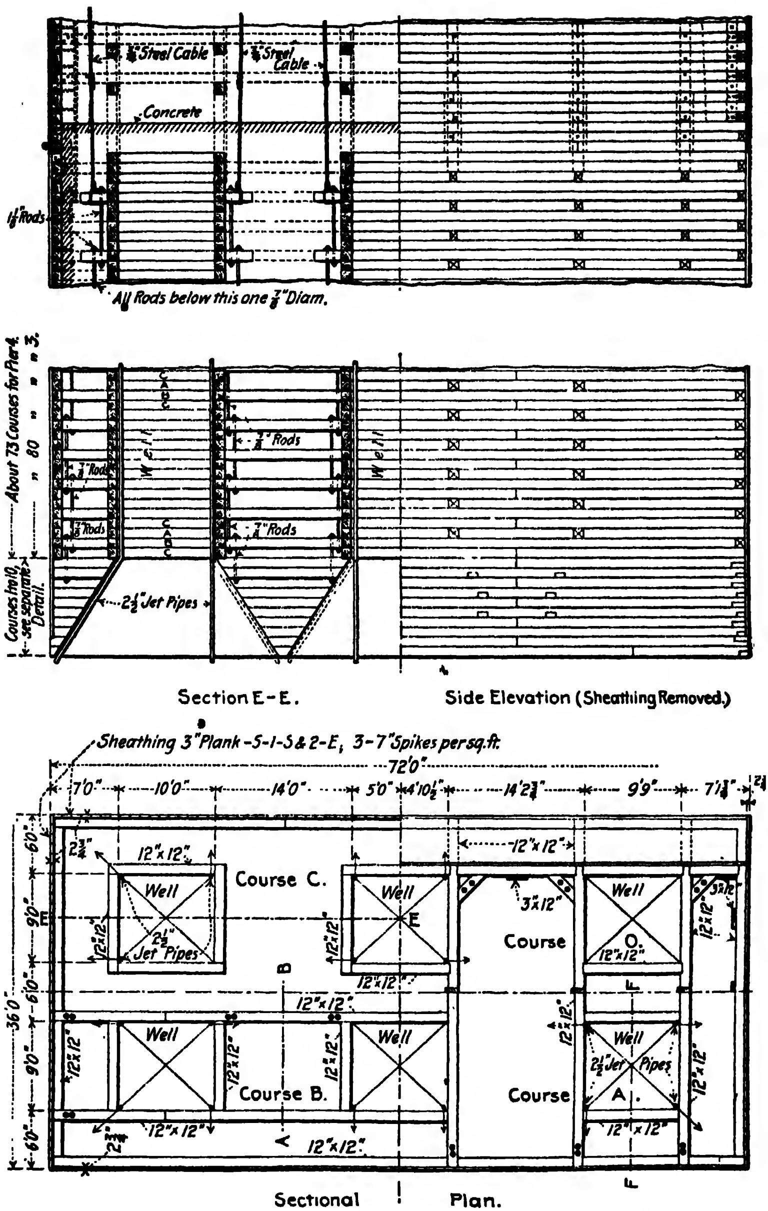


FIG. 91b.—Open Caisson and Cofferdam for the Oregon-Washington Railroad and Navigation Company's Bridge at Portland, Ore.

and one longitudinally. The widths at the top were 6 feet for the longitudinal walls and bulkheads, 7 feet for the transverse walls and 14 feet for the transverse bulkhead. In all cases

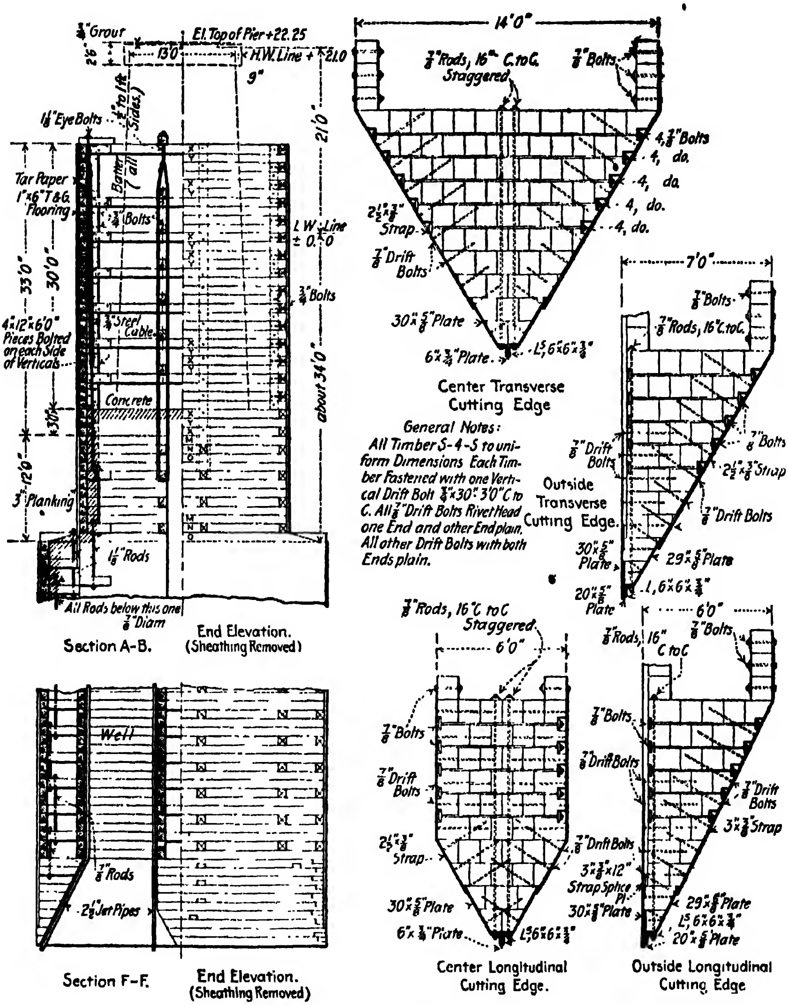


FIG. 91c.—Open Caisson with Dredging Wells and Superimposed Cofferdam.

the cutting edges were reinforced with steel angles as shown in the drawings.

A 33-foot cofferdam surmounted the caisson, the walls being of the same construction as those of the caisson, except that the sheathing consisted of 1-inch tongue-and-grooved material, with tar-paper between it and the large timbers. The cofferdam was braced with horizontal 12- by 12-inch timbers running both longitudinally and transversely, and bearing against vertical 12- by 12-inch timbers, which, in turn, took bearing against the walls of the cofferdam.

Before sinking the caisson, borings made around the perimeter of the crib showed that the surface of the good bearing stratum was on a considerable slope, a difference of 22 feet being found for opposite diagonal corners. To level this off, pipes were sunk and holes drilled to about 2 feet below the lowest elevation of the top of this cemented gravel stratum. Dynamite was placed in these holes and exploded, and in this way the hard material was broken up through 50 feet of gravel and sand, before any excavation had been made. When the caisson reached this cemented gravel the latter was easily removed with orange-peel buckets working through the dredging wells. The depth to which the caisson was sunk was about 130 feet below low water, or 151 feet below high water.

In the new Thames River bridge of the New York, New Haven and Hartford Railroad at New London, the lower part is of concrete and the upper part of timber, as shown in Fig. 91*d*. An enlarged view of the lower part is shown in Fig. 108*c*, which also shows the steel protection of the cutting edge.

The Poughkeepsie bridge, which spans the Hudson River at Poughkeepsie, N. Y., was the first structure in America to be founded on deep, wooden, open caissons, and these caissons are among the largest and deepest that have ever been placed. In some details, such as filling the pockets with gravel instead of with concrete, and using a removable cofferdam on grillage on top, instead of the ordinary cofferdam, they differ materially from what is now standard practice.

The caisson for the longest pier was 60 by 100 feet in plan at the bottom. The sides were vertical for a height of 40 feet from the bottom, and from that point they were battered to

give a width of 40 feet at the top. The height of the caisson was 104 feet, and, according to BAKER's Masonry Construction, its cutting edge rests on a bed of gravel 134 feet below high water, thus making the top of the caisson 23 feet below low-water level.

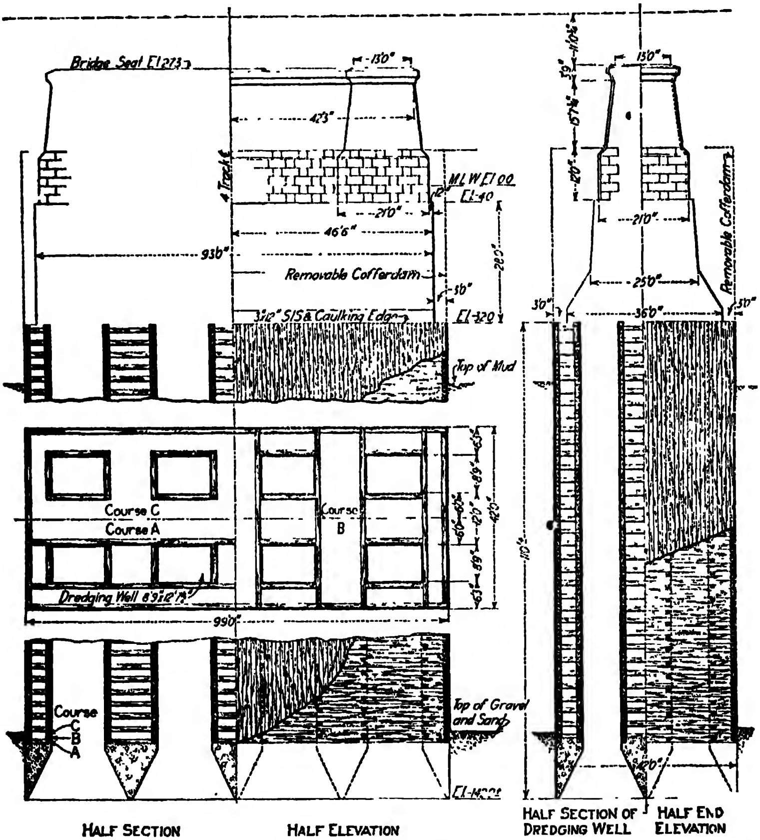


FIG. 91*d*.—Timber Open Caisson with Dredging Wells of New Thames River Bridge at New London, Conn.

Fourteen dredging wells, 10 by 12 feet in plan, were formed by one longitudinal and six transverse walls. The exterior and longitudinal walls were built solid and of a triangular section for a distance of 20 feet from the bottom; at this point the thickness was 10 feet for the side walls, 9 feet for the end walls, and

16 feet for the middle longitudinal wall. Above this they were hollow, each wall dividing into two 2-foot walls with a hollow space between, the latter forming the filling pockets. The six transverse walls were made 2 feet thick and extended from the cutting edge to the top of the caisson. All walls were made of 12- by 12-inch material laid horizontally.

The sinking was accomplished by dredging through the wells with a clam-shell bucket and by filling the pockets with gravel. Care had to be taken that the pockets were completely filled before the top of the caisson reached water-level, in order that a layer of 12- by 12-inch timber could be laid over these pockets. The remainder of the dredging was done with the top of the wells submerged. On the caisson reaching the desired depth the wells were filled with concrete.

A grillage 6 feet thick was then constructed and temporary walls 6 feet high were built on it. This was floated to position over the caisson and masonry built up in it. As soon as the weight of the masonry was sufficient to cause the walls to become submerged, the latter were removed, the masonry by this time being well above the surface of the water. More masonry was added until the grillage came to a bearing on the caisson, after which the remainder of the masonry pier was built.

ART. 92. CONSTRUCTION WITH METAL

The use of iron and steel shells for open caissons of other than the cylindrical form has not found favor in America; on the other hand, English engineers have made extensive use of this type. A general statement may perhaps be made that where the American engineer will use a pneumatic caisson of wood his English brother will use an open caisson of metal, either steel, wrought iron or cast iron.

The two advantages possessed by this type of caisson are: first, the speed with which the caisson may be built, and second, the small space occupied by the metal, thus leaving a maximum amount of space for the concrete filling. The disadvantages are: first, the cost of the caisson, especially where

the metal has to be transported long distances, and second, the lack of permanency of the metal.

One of the most notable examples of the use of open caissons with dredging wells is that of the Hardinge bridge across the Lower Ganges, 120 miles above Calcutta, opened in 1915. The

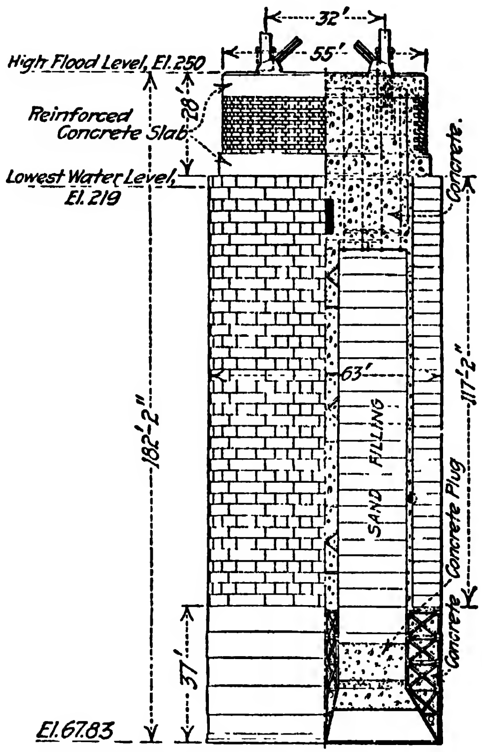


FIG. 92a.—Metal Open Caisson with Dredging Wells for Hardinge Bridge.

deepest bridge foundations on record—150 to 160 feet below low water and 190 feet below high flood level—are found here.

In plan the caissons have semi-circular ends and straight sides, being 35 by 63 feet. As shown in Fig. 92a, the caissons contain two dredging wells each 18½ feet in diameter. The well curb is built of steel and is 15 feet 7 inches high. It is continued upward as a caisson, the distance varying in different

caissons depending on the depth of water. The space between the walls is filled with concrete. Above this the dredging wells are lined with steel to give water-tightness and added strength and also to serve as a form for the mass concrete. Above the steel-frame part, the walls consist of molded concrete blocks weighing about 6 tons each, these blocks being carried up to low-water level. After the caissons were sunk to their final depth, the bottom and the top of the wells were plugged with concrete and the space between filled with sand. The caissons rest on sand, the pressure being 9 tons per square foot, allowing for buoyancy but not for skin friction.

Another notable example of the use of open caissons with dredging wells in bridge foundations is that of the substructure of the Hawkesbury bridge in southwestern Australia, where the caissons were sunk to a maximum depth of almost 162 feet below high water, the range of tide being 7 feet.

The caissons of the Hawkesbury bridge were oblong in plan with rounded ends, the length being 48 feet and the width 20 feet. The lower 20 feet was splayed out to form a tapered shoe 2 feet wider all around the bottom. Along the center line, parallel with its length were three wrought-iron dredging tubes, 8 feet in diameter and 14 feet apart on centers, strongly braced to the sides of the caisson and to each other. At the bottom these wells splayed out in the form of a trumpet mouth to meet the outer skin and each other in a cutting edge made of steel. Between these wells and the sides of the caisson were pockets to be filled with concrete as the caisson sank.

Sections of steel for the sides and of wrought iron for the dredging tubes were added as the caisson sank, the sinking being effected by dredging out the material under the caisson through the wells and by filling the pockets with concrete. All caissons were bedded on firm sand which was overlaid with mud and silt of varying depth, that at pier 6 being 108 feet with a depth of water at low tide of 47 feet. As soon as the caissons were firmly bedded in the sand, the wells were filled with concrete and the pier masonry was started at a depth somewhat below low water.

The experience gained in sinking these caissons showed that it is not advisable to splay out the outside walls of a caisson. If this is done, the guiding effect of the surrounding material is largely lost, and a very troublesome condition obtains when on one side the earth is firmer than on the other, thus standing for some time after the other has fallen in, as a consequence of which the caisson is forced out of position.

In the construction of the Dufferin bridge piers over the Ganges River at Benares, the caissons, which were elliptical in form and 65 by 29 feet in plan, had their pockets filled with brickwork, only the lower 6 feet being of concrete. In the Hoogly bridge caissons the outer skin was vertical all the way from the bottom, while the three dredging chambers extended across the structure and occupied the two semi-circular ends as well as the central portion, the remainder forming two pockets each 15 feet in width. Sinking weight was obtained by concreting the two pockets and by a brick lining, 3 feet wide, around the semi-circular ends, this lining resting on horizontal shelf angles spaced 4 feet apart vertically. The upper part of the central well also had a brick lining.

ART. 93. CONSTRUCTION WITH CONCRETE

The use of concrete open caissons, usually reinforced, is rapidly increasing, and without doubt this type will replace the other forms in the future. The concrete type offers many advantages over those made of wood or metal: it does away with the uncertainty of future decay or corrosion; its greater specific gravity, compared with wood, makes less weighting necessary in sinking than when the wooden type is used; its cost will compare favorably with the other forms; and with its concrete filling it forms a monolithic foundation, far stronger and better than any combination structure. All caissons, whether of wood, steel or concrete, furnish essentially a concrete foundation for the pier, the shell being the only part which may not be of concrete. The one possible disadvantage of the concrete caisson is the greater time element involved,

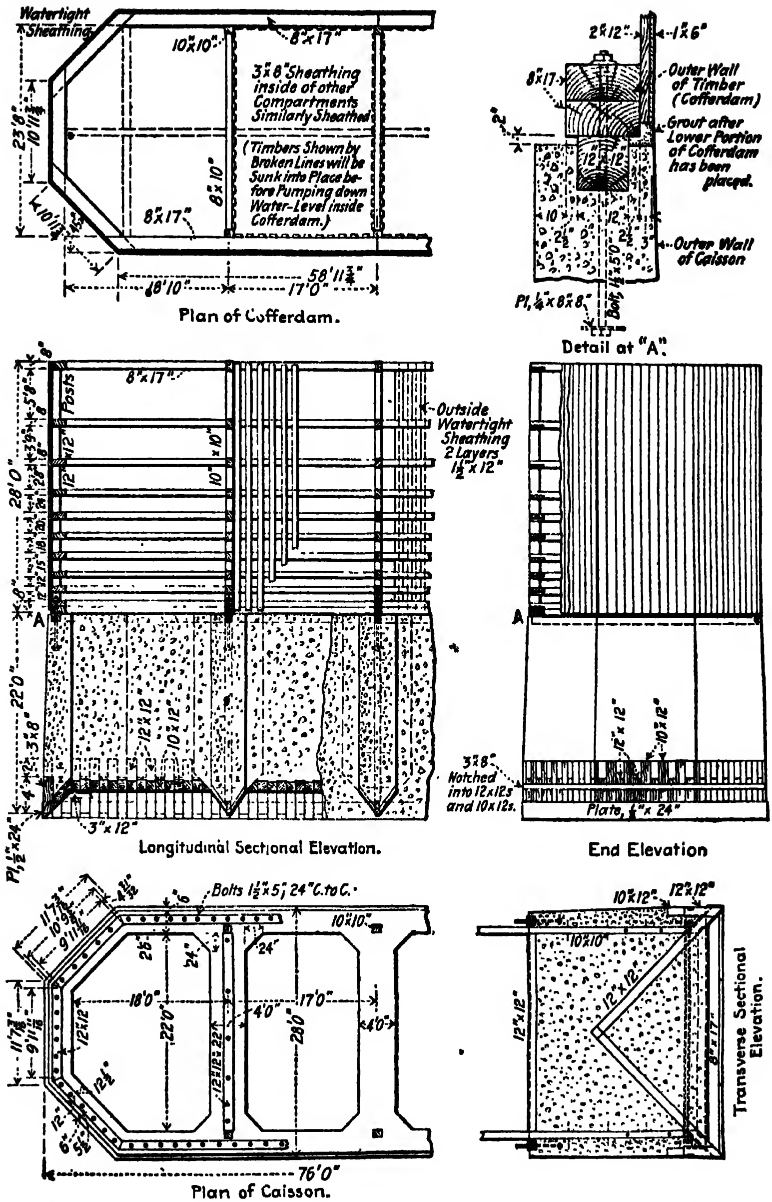


FIG. 93a.—Open Caisson and Cofferdam for Pier 2 of Southern Pacific Railroad Bridge over American River near Elvas, Cal.

since the shell cannot be sunk until it has hardened for some time, but if the work is properly laid out this element will be of little moment.

One of the piers of the American River bridge, of the Southern Pacific Railroad, was founded upon a concrete open caisson, the details of which are shown in Fig. 93*a*, while Fig. 93*d* shows the forms used for this caisson. The physical conditions at the site were peculiar. The hardpan, which was covered by a bed of cobblestones and small boulders, was from 40 to 60 feet below water-level. When the water was low, a deposit of fine gravel and silt overlaid the cobbles, but when freshets came this gravel and silt were entirely scoured out. The reinforced-concrete caisson used for pier 2 was 28 by 76 feet in plan and 22 feet in height. The side and end walls were 3 feet thick, while the three cross-walls which divided the structure into four compartments, were each 4 feet thick. The cutting edges were made by beveling the walls, and each of these cutting edges was reinforced with angles and plates. The walls were reinforced near the bottom with old steel rails laid horizontally.

The caisson was sunk during a time when the bed of the stream was dry, the soil being first excavated down to ground-water level, after which the forms were placed and the concrete poured, the entire caisson being built before any sinking was done. Sinking was effected by excavating the material from the compartments with an orange-peel bucket. When down to about ground level a cofferdam was added. When the cutting edge reached the stratum of cobbles and boulders, sinking operations were stopped and the compartments of the caisson were filled with concrete, after which the pier was built in the cofferdam. The process of sinking was slow, owing to the fact that no weighting was done.

[1]"The concrete caisson was surmounted by a timber cofferdam, 28 feet high, which had tiers of horizontal rangers and braces attached to 10- by 10-inch uprights anchored to the concrete. The lower courses of rangers were also anchored to the latter, a special joint being formed between them and the top

[1] Engineering Record, vol. 62, page 232, Aug. 27, 1910.

of the walls to prevent leakage at this point. The sides of the timber cofferdam were formed by two courses of 1½-inch ship-lap nailed closely to the rangers. The inside walls of each compartment were also sheathed with 3- by 8-inch vertical pieces, spaced 8 inches apart."

One of the most notable examples of the use of the all-concrete open caisson is that for the Beaver bridge of the Pittsburgh and Lake Erie Railroad. They were among the first of the all-

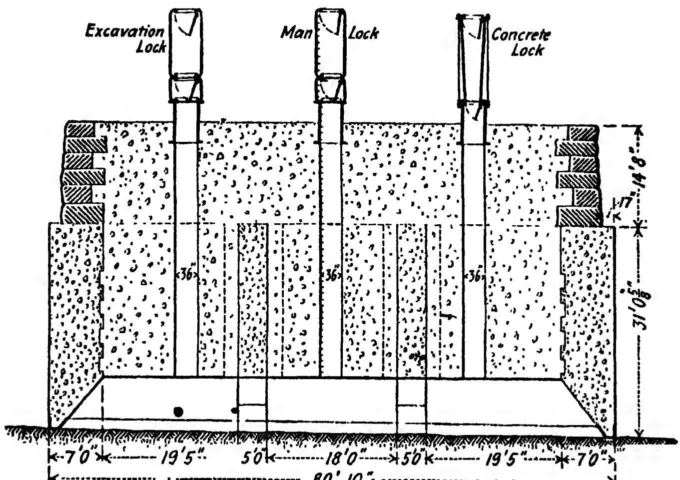


FIG. 93*b*.—Caisson for Pittsburgh and Lake Erie Railroad Bridge, Beaver, Pa.

concrete caissons, being placed in 1908, and the largest to that date. Two novel features were: first, the very considerable thickness of walls, making it possible to dispense with reinforcement in the concrete; and, second, the use of the pneumatic process in bedding the caissons. The pneumatic feature of these caissons is described in Art. 102.

At the site of the piers the ordinary depth of water was about 7 feet, with bed rock about 38 feet below the bed of the river. The overlying material was mostly sand and gravel, with many boulders scattered throughout the mass.

The caissons, views of which are shown in Figs. 93*b* and *c*, consisted of a concrete shell 7 feet thick, this thickness being

maintained from the top to a point about 9 feet above the shoe, at which elevation it tapered to an 8-inch cutting edge. This cutting edge or shoe was formed by an 8- by 8- by 5/8-inch angle and a 21- by 1/2-inch bent plate, the vertical leg of the angle extending upward on the outer face of the caisson, while the bent plate had its inclined leg along the inner face of the wall. Rods 7/8 inch in diameter and 10 feet long, extending up into the concrete, held the shoe in place. There were two cross-walls each 5 feet thick to stiffen the caisson, and these extended from the top of the caisson to about 3 feet above the shoe, a trapezoidal portion at the bottom of each wall about 6 feet in height being omitted.

Rectangular cofferdams were first constructed around the site of each pier and these were unwatered to permit building the forms for the caissons. On completion of the forms a 6-foot depth of concrete was deposited in the same and allowed to harden before more was added.

Sinking was accomplished for the most part by dredging through the three wells with orange-peel buckets When it was possible to keep the water down by pumping, the spoil under the shoe of the caisson was removed by hand to within reach of the buckets. Later, when boulders were encountered under the cutting edge, they were removed by divers When the caisson showed a tendency to stick, the water-jet was used. When within about 16 inches of the rock the pneumatic process was brought into use as explained in Art. 102.

That the omission of reinforcement in a structure like the one just described may prove a costly mistake was shown in the caissons for the North Side Point bridge, across the Allegheny River, Pittsburgh, Pa. The caisson for the river pier had dimensions of 23 by 83½ feet in plan, with four rectangular wells, 9 by 10 feet, and spaced 19 feet center to center. When the pier had been sunk to a depth of 17 feet below river bottom, a transverse crack developed at about midlength and extended from the top to the river bottom, as shown at the left in Fig. 90*e*. The cracking was due to the unequal dredging in the different wells, causing the weight of the caisson to bear chiefly

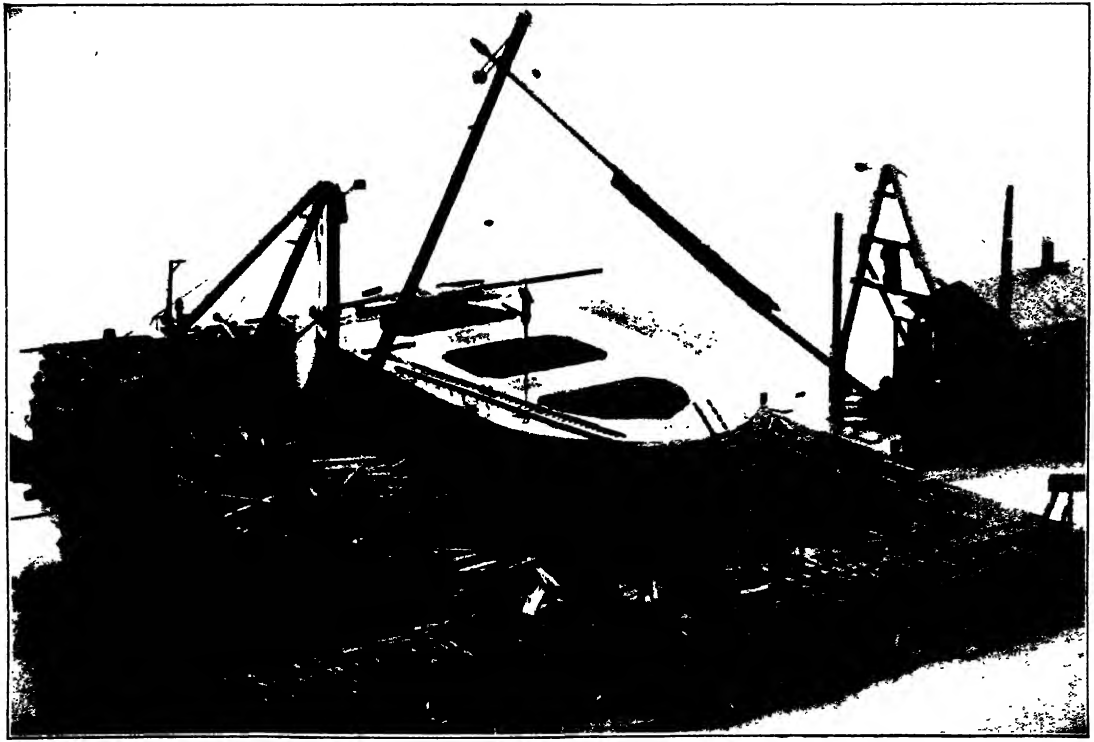


FIG. 93c.—Reinforced-Concrete Open Caisson for Pier 3 of the Pittsburgh and Lake Erie Railroad Bridge over the Ohio River at Beaver, Pa. September 28, 1908.

(Facing p. 300.)

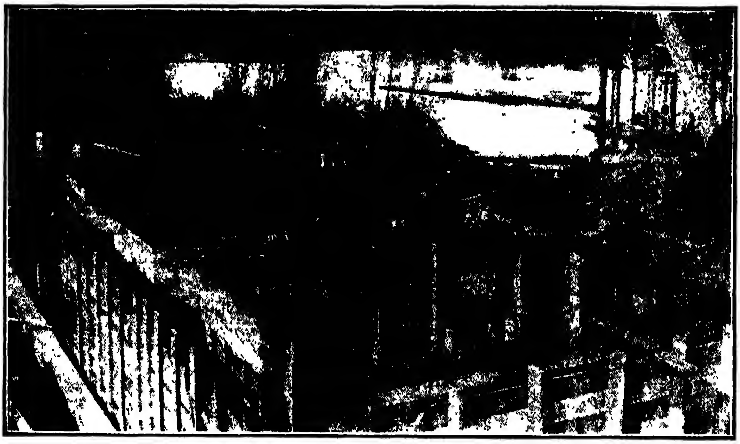


Fig. 93*d*.—Forms for Open Caisson of Concrete. See Fig. 93*a*.

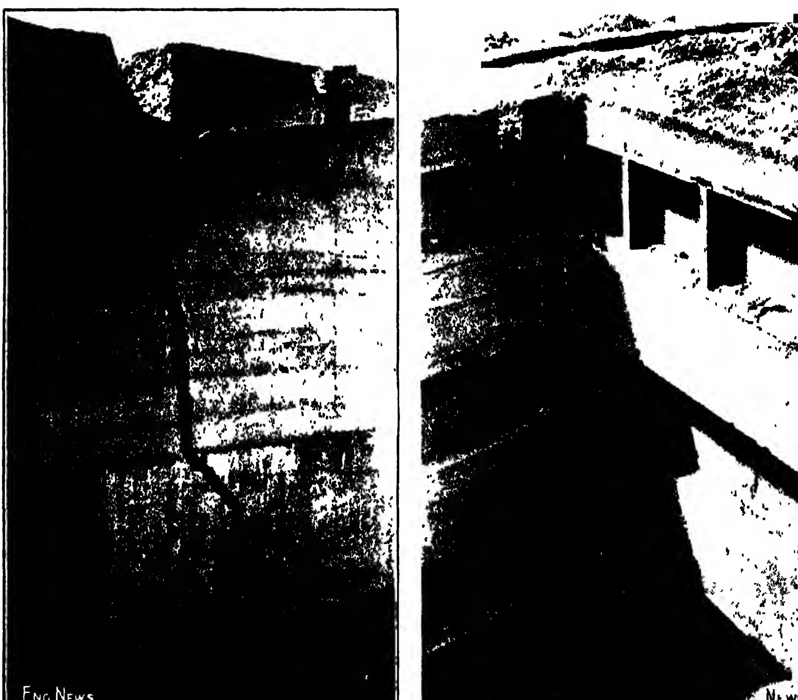


Fig. 93*e*.—Cracks in Caisson of Concrete without Reinforcement.

at midlength. The cracked caisson was blasted to pieces and removed. The one placed afterward had smaller wells, and was reinforced with longitudinal rods.

The concrete caisson for the Camden anchorage of the Delaware River bridge, sunk in 1922, was 40 by 140 feet in plan,

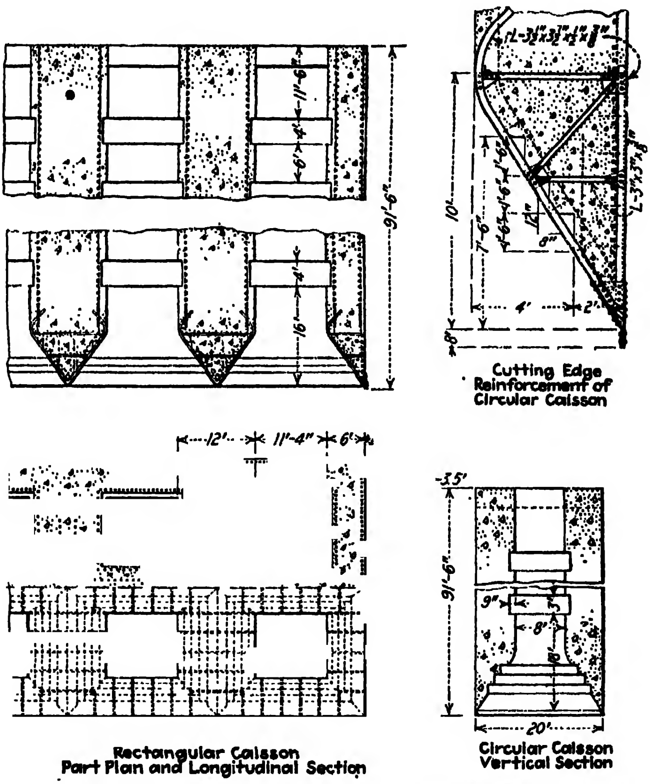


FIG. 93*f*.—Camden Anchorage Caissons, Delaware River Bridge.

or 5600 square feet in area, making it the largest of concrete to date (1915). This caisson, shown in Fig. 93*f*, has 12 wells 11⅓ feet square in two longitudinal rows. The side and end walls are 6 feet thick, the longitudinal wall 8 feet and the transverse walls 12 feet. A large amount of reinforcement in the form of structural shapes and bars was used in this design.

For an example of caissons for buildings which are sunk as open caissons and can be transformed into the pneumatic type, see Engineering Record, vol. 63, page 185, Feb. 18, 1911.

The first all-concrete open caisson used for bridge foundations in this country is probably the one for pier D of the Thebes cantilever bridge. This caisson is 19 by 38 feet in plan and was placed in the winter of 1902–1903. An open caisson (which was not constructed of either timber or metal) was first sunk in this country in 1898 by the Dravo Contracting Co., at Neville Island near Pittsburgh and was used for a pump well.

ART. 94. SINKING OPEN CAISSONS

Open caissons are built and placed in position in a manner similar to that of pneumatic caissons, described in Art. 105.

There are five methods used in sinking open caissons: first, removing the material from within the caisson; second, weighting the structure; third, using the water-jet; fourth, driving down the caisson; and, fifth, pulling down the caisson.

The first method, which represents the fundamental idea of the open caisson, is always employed. For the small caisson, to be sunk but a few feet, as practiced by the natives of countries in the Far East, excavating is done by baskets carried down, filled and brought up by divers. The modern method consists in pumping, or in dredging out the material with buckets.

The mud and sand pump, the principle of which is described in Art. 110, is used where the material is largely silt, or other soft material. Figure 94*a* illustrates the pump used on the Fraser River bridge, at New Westminister, B. C. This pump, or ejector, which was operated by a hydraulic jet, at a pressure of 125 pounds per square inch, could handle anything with a diameter of less than 3 inches. The dimensions of the pump are shown in the illustrations. The top of the pressure pipe was fitted with a ball-and-socket joint and a 90-degree bend with an enlarger, to which were connected three lines of 2½-inch fire hose. A separate hydraulic jet, having a ¾-inch hole at the bottom and five ⅜-inch holes in nearly vertical planes on

the circumference of a circle and a few inches above the bottom, was used to agitate the material around the intake of the ejector.

The most common method of removing material is by dredging with an orange-peel or a clam-shell bucket. Where a layer of stiff clay is met, it may be broken up by sending divers down

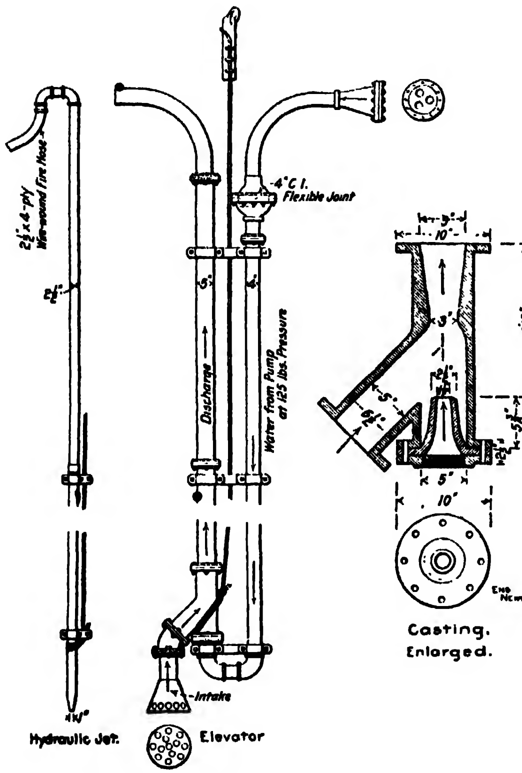


FIG. 94a.—Details of Hydraulic Ejector.

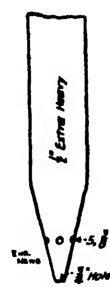


FIG. 94b.—Tip of Water-Jet.

to blast it to pieces, or it may sometimes be broken up by dropping down long steel rails vertically, which sink into the clay and tear it up in tipping over. A line is attached to the rails to withdraw them.

The most economical way to weight caissons is by making use of the permanent filling, and for this reason, where the size of the

caisson makes it possible, a double wall should always be used. Temporary weighting, as with rails laid on top, etc., is always expensive on account of the time and labor involved, as well as on account of obstruction to the dredging operations.

The rapid and successful sinking of the Hawkesbury bridge caissons (Art. 92) was largely due to their being designed to carry a large mass of concrete between the outer and inner shells, which was placed during sinking.

The water-jet (Art. 18) is always a useful adjunct in caisson-sinking operations. By using the same freely around the cutting edges and along the sides the frictional resistance is considerably decreased. Another advantage is that it tends to wash the material toward the interior of the caisson, where it can be picked up by the dredging buckets, which have previously made a hole in the center.

It is possible to drive caissons only when they are small and even then only light blows with the hammer may safely be given. Pulling the caisson down may sometimes be employed to advantage, if it is possible to drive piles around the outside and attach tackle to them and to the sides of the caisson.

CHAPTER VIII

PNEUMATIC CAISSONS FOR BRIDGES

ART. 95. THE PNEUMATIC PROCESS

The use of the plenum pneumatic process for founding deep piers is a good example of the application of scientific principles to foundation work. A pneumatic caisson may be defined as a structure, open at the bottom and closed at the top—in other words, an inverted box—in which compressed air is utilized to keep the water and mud from coming into the box, and which forms an integral part of the foundation.

The caisson, which is usually not over 6 feet high in the working chamber, is surmounted by a crib and cofferdam, the former, with the exception of one or more vertical wells, called shafts, being filled with concrete as the caisson sinks. This concreting, together with the excavating done in the working chamber, as the interior of the caisson is called, effects the sinking of the latter.

The working chamber must be practically air- and water-tight, and yet there must be an opening for men to enter and leave the chamber, as well as an inlet and outlet for materials. These openings are provided by vertical shafts and air-locks. The shafts, which extend from the roof of the caisson to a point well above the top of the crib and the level of the water outside, are usually of a circular or oval section and from 2½ to 4 feet in maximum diameter. In the shafts, at the bottom, top, or between these two points, are placed the air-locks, they being air-tight chambers, often simply a part of the shaft itself, fitted with two doors, one of which leads to the working chamber and the other to the open air.

The most pronounced advantage of the pneumatic-caisson as compared with the open-caisson process lies in the fact that the

engineer has more control over the work, having a better opportunity to sink the caisson vertically, to remove large boulders, sunken logs, etc., from under the cutting edge; the foundation bed can be properly prepared and personally inspected; and, lastly, the concrete filling of the working chamber is deposited in air, thus giving a superior foundation. Another point, which is sometimes of great importance in placing foundations for buildings, is that the soil about the caisson is not so liable to be disturbed when the pneumatic process is used. The one disadvantage of this process is that the men have to work under an air pressure which is sufficient to balance the pressure of the surrounding water in addition to atmospheric pressure, or practically the full hydrostatic head from the cutting edge to the water surface.

For depths from about 30 to 110 feet this type of caisson is extensively employed. For depths less than 30 feet the cofferdam process is usually, but not always, a more economical method of placing the foundation, while for depths greater than about 110 feet, corresponding to a pressure of over three atmospheres above the normal, the open-caisson method must be employed, since men cannot work advantageously under such high pressures. Probably the minimum depth for which a pneumatic caisson was used was for an elevator pit near a high building in New York in 1909, where the depth below water level was only 6 feet. The presence of quicksand made open excavation methods too risky.

The maximum depth (1915) below water surface for bridge caissons is 115 feet, this depth being reached in placing the caissons of the Raritan River highway bridge at Perth Amboy in 1924. The maximum pressure used was much less than for some other bridges, however, as the material sunk through was light clay and sand.

Among other notable examples of deep immersions are the Metropolis bridge, 113.2 feet and 51-pound air pressure; the St. Louis Municipal bridge, 112 feet and 50-pound air pressure; the Boulak bridge over the Nile at Cairo, 111.5 feet; the Lexington bridge over the Missouri, 110 feet and 52-pound air

pressure; the St. Louis arch bridge, 109.7 feet; the Williamsburg bridge (New York), 107.5 feet; and the Memphis bridge, 106.4 feet.

The caissons used in sinking a mine shaft near Deerwood, Minn., were sunk to a depth of 123 feet below ground-water level and 130 feet below the ground surface. The maximum pressure used was 52 pounds per square inch.

The first use of compressed air was made by TRIGER, a French engineer, in sinking a shaft in 1839. This method was used for placing bridge pier foundations in Europe in 1851 and in the United States in 1852, the latter being for the foundations of bridges over the Pedee and Santee rivers. Here the caissons consisted of cast-iron cylinders, called pneumatic piers, which formed both the working chambers and sections of the piers.

The St. Louis arch bridge was the first in this country to be founded on large pneumatic caissons, its east abutment caisson, which had a maximum immersion of 109 feet 8½ inches, being sunk in 1870. The second bridge in this country to be founded on large pneumatic caissons was the great Brooklyn suspension bridge, which, in its New York tower caisson, sunk in 1871, has the largest pneumatic caisson ever placed for a bridge foundation. It was 102 by 172 feet in plan and was sunk to a depth of 78 feet below high-water level.

Following is a list of the largest pneumatic caissons built to date (1921):

Date	Bridge	Dimensions	Area, square feet
1871	New York pier, Brooklyn bridge......	102 by 172	17,544
1898	Alexander III bridge, Paris...........	110 by 145	15,850
1901	Manhattan bridge, New York........	78 by 144	11,232
1922	Delaware River bridge	70 by 143	10,610
1910	Quebec bridge......................	55 by 180	9,900
1914	Metropolis bridge...................	60½ by 110½	6,655
1869	East abutment, St. Louis arch bridge...	72½ by 82 (hex)	6,000

The pneumatic-caisson process has been widely used in America and on the European continent. As a class, English

engineers have apparently shown some aversion to it, and in many cases, where it seems to have been the preferable structure on account of the presence of boulders and logs, the open-caisson process was used. American engineers have developed the wooden caisson to a high state of perfection, but at present (1925), owing to the high price of timber, the tendency is toward the use of more reinforced concrete and steel. In Europe the metallic form of pneumatic caisson has been extensively used.

To give some indication of the progress made in the science and art of foundation construction it is interesting to note that the cost per cubic yard of the substructure of the Municipal bridge at St. Louis is only 29.6 percent of the corresponding cost of that of the St. Louis arch bridge, which is located about a mile above it, and 50.8 percent of that of the Memphis bridge. The substructures of these three bridges were completed in 1911, 1871 and 1891 respectively. In this comparison the approaches are excluded. As previously noted, the three bridges have deep foundations.

The contract price for the caisson work of the Municipal bridge was $27 per cubic yard below the cutting edge, and $12.90 per cubic yard from the cutting edge to the top of the crib. The timber caissons of the Portland, Ore. bridge placed in 1911 cost $14 per cubic yard in place.

ART. 96. CAISSON ROOF CONSTRUCTION

TIMBER ROOFS.—The design of the roof has always been largely a question of judgment, as it is almost impossible to analyze the stresses. The tendency in roof construction has been constantly to decrease the thickness of the timber roof and consequently its cost. When concrete superseded stone masonry as a filling for the crib, a considerable decrease in the thickness of the roof was made possible on account of the strength of the concrete. A more generous use of bulkheads and the arrangement of the bracing above and below the deck to act as trusses also aided in securing a thinner roof. At

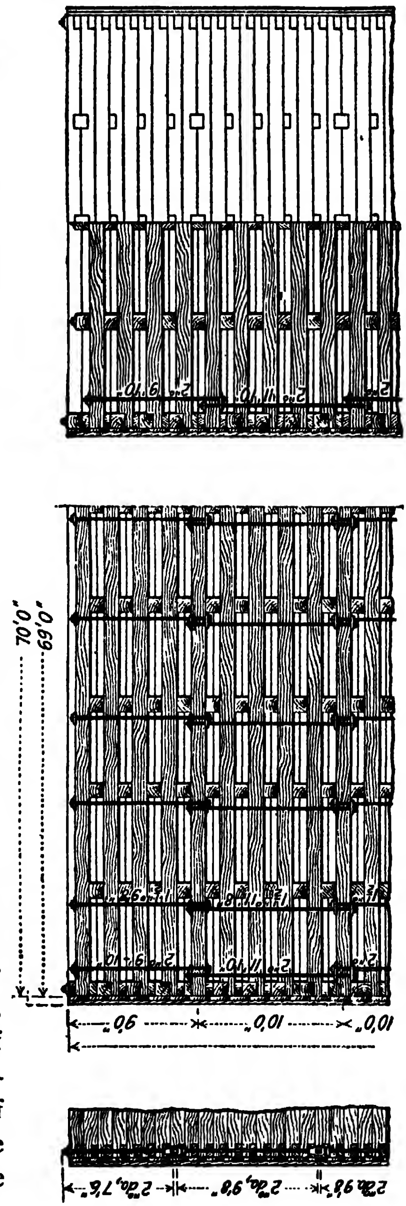
present many caissons do away with a permanent timber roof almost entirely by reinforcing the concrete filling of the crib.

The roof is usually made with layers of 12- by 12-inch timbers, sheathed on the lower side with 2- or 3-inch planks. Sheathing may also be used between the courses of large timbers. The various courses run in different directions; if the roof is of a two-course thickness, both courses may run transversely, while if it has three courses the lower and upper courses run transversely and the middle course longitudinally. All calking of the air chamber is done from the inside of the working chamber, against the air blowing out, while the outside planking is calked from the outside, to prevent the water from getting in.

The roof of the 102- by 172-foot caisson for the New York tower of the Brooklyn bridge was composed of a solid mass of squared timbers, 22 feet thick, all timbers being 12 by 12 inches in section, and thoroughly drift-bolted together. This is the thickest roof that has ever been used.

The roof of the 31- by 79-foot rectangular caisson for the old piers of the Baltimore and Ohio Railroad bridge at Havre de Grace was composed of eight thicknesses of 12- by 12-inch timbers, the courses alternating in direction, some running longitudinally, other transversely and still others diagonally. The lower surface was sheathed with 3- by 12-inch planks. This form of roof is typical of a number of caissons built under the direction of WILLIAM PATTON, who was an extremist in respect to thick roofs.

The roof of the east-abutment caisson of the St. Louis arch bridge was only 4 feet 10 inches thick, the upper three layers being composed of 16- by 16-inch timbers. The shape of this caisson was an irregular hexagon, with extreme dimensions of 82 by $72\frac{1}{2}$ feet. This comparatively thin roof was made possible by the use of two wooden bulkheads below the roof and two iron girders above, the latter running at right angles to the former, and all supporting the roof. The upper surface of this roof was covered with plate iron, while in the Brooklyn bridge caissons the under side was covered with wrought-iron plates; in both cases this was done to obtain an air-tight roof.



It was a very expensive method, since oakum calking is sufficient. But in the Brooklyn bridge caissons it was done for the added purpose of fire protection, for in those early caissons torches were used for lighting purposes, and as there was always a considerable amount of air escaping between the timbers the danger of fire was very great.

In recent years the tendency has been to use more courses of 3-inch sheathing, usually tongue-and-groove, in order to get a more nearly air-tight roof. As shown in Fig. 96*a* the roof of the caisson for pier 4 of the Bellefontaine bridge, built in 1892, consisted of two courses of large-size timbers, between which were placed two courses of sheathing, laid diagonally. The lower side of the roof was also lined with sheathing. Another notable feature of this roof, which is characteristic of many built by Geo. S. MORISON, is the relatively thin roof used. This was made possible by connecting the roof to the bracing timbers of the crib above by means of tie rods.

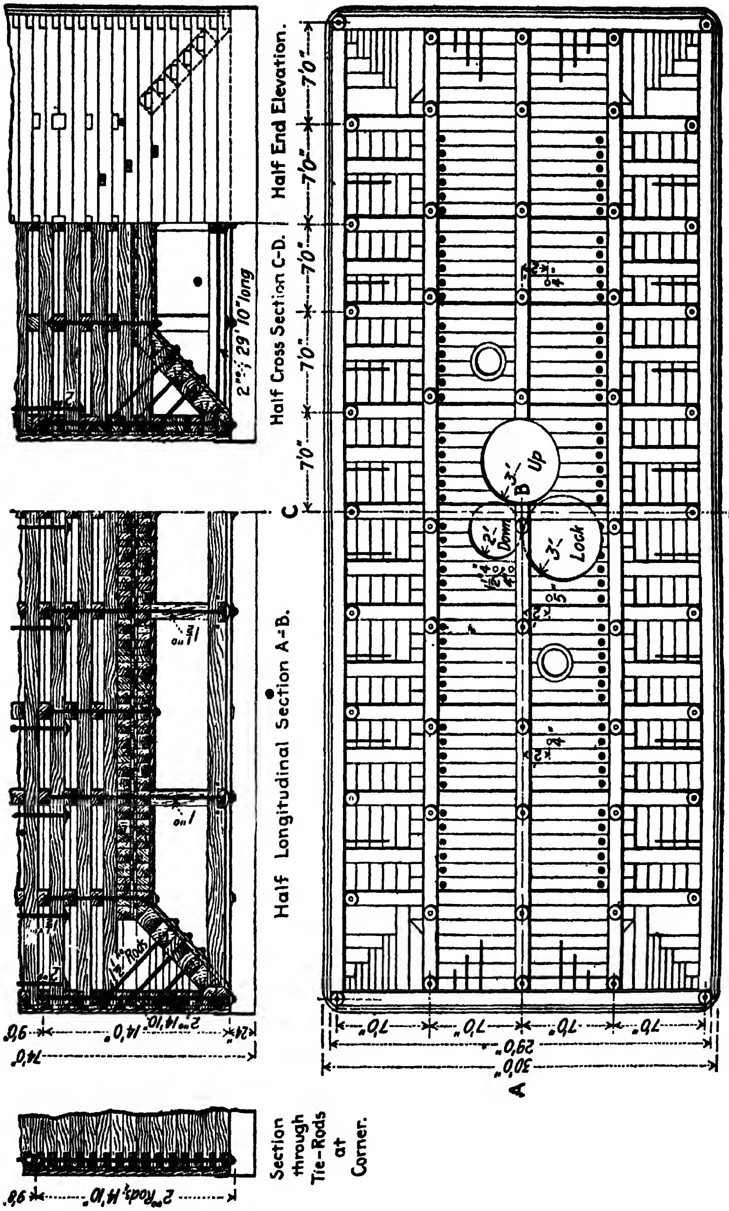


FIG. 96a.—Pneumatic Caisson for Pier IV of Bellefontaine Bridge. Designed in 1892.

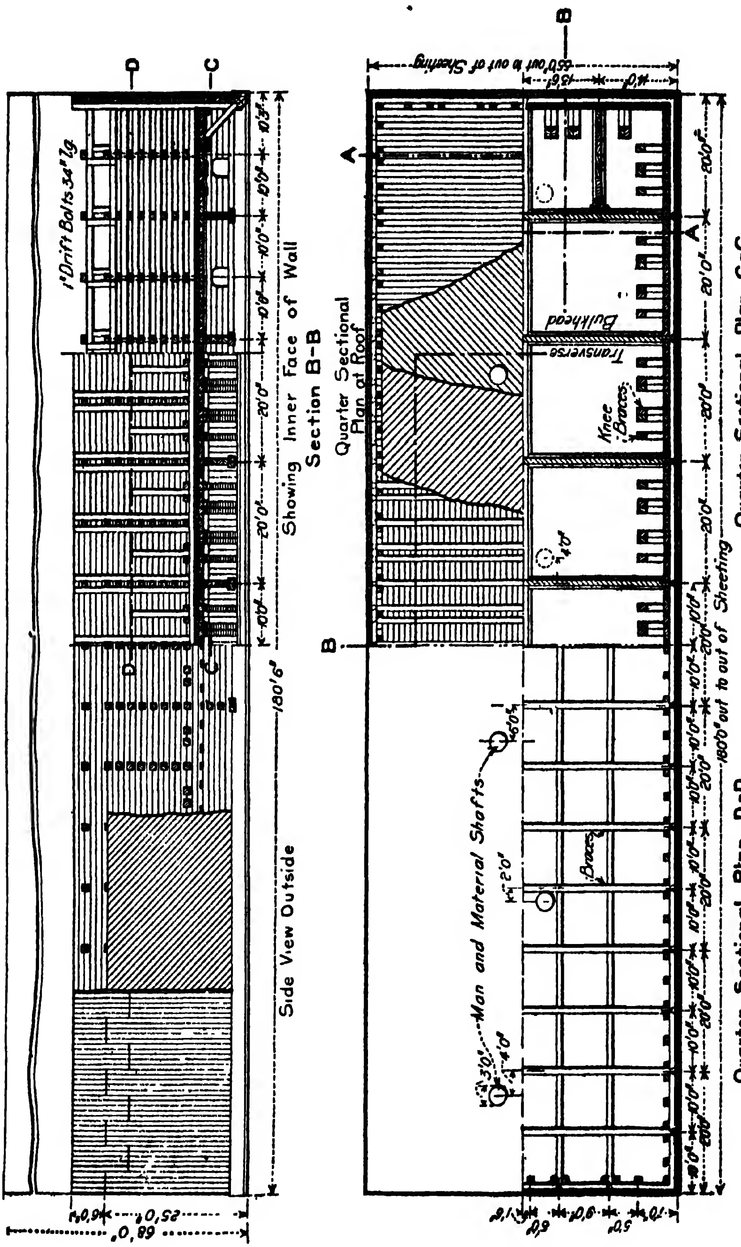


FIG. 96b.—Pneumatic Caisson for South Main Pier of Second Quebec Cantilever Bridge.

As shown in Figs. 96*b*, *c*, and *d*, the roof of the south main pier caisson of the second Quebec bridge consisted of one solid course of longitudinal and one solid course of transverse timbers, separated by two crossed courses of diagonal 3-inch

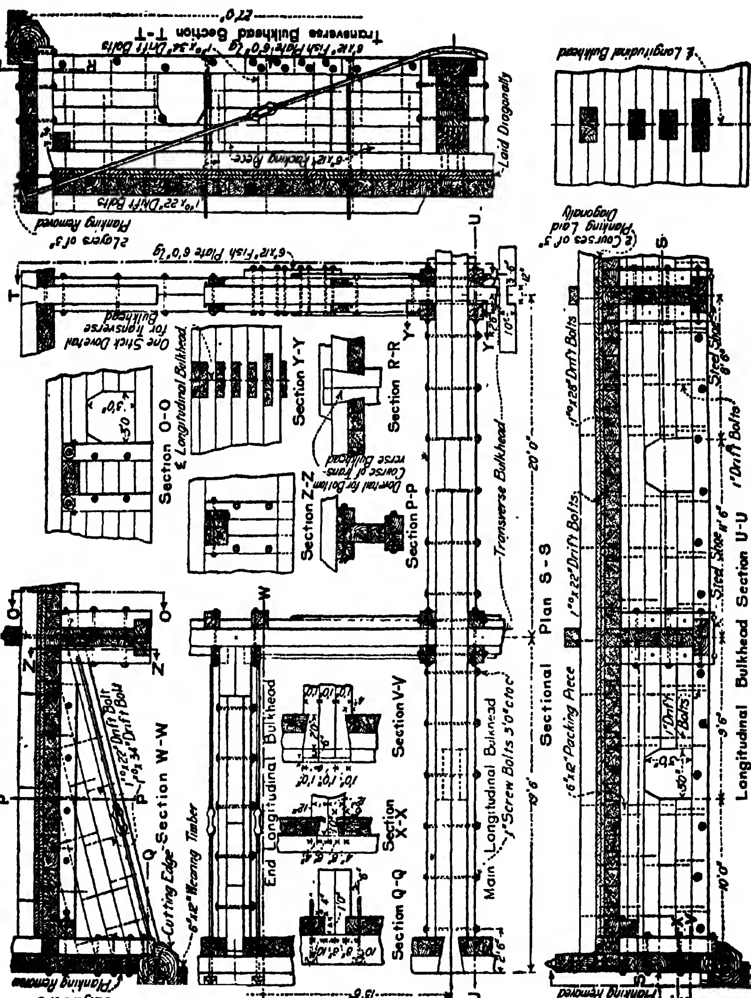


FIG. 96*d*.—Details of Pneumatic Caisson for Quebec Bridge.

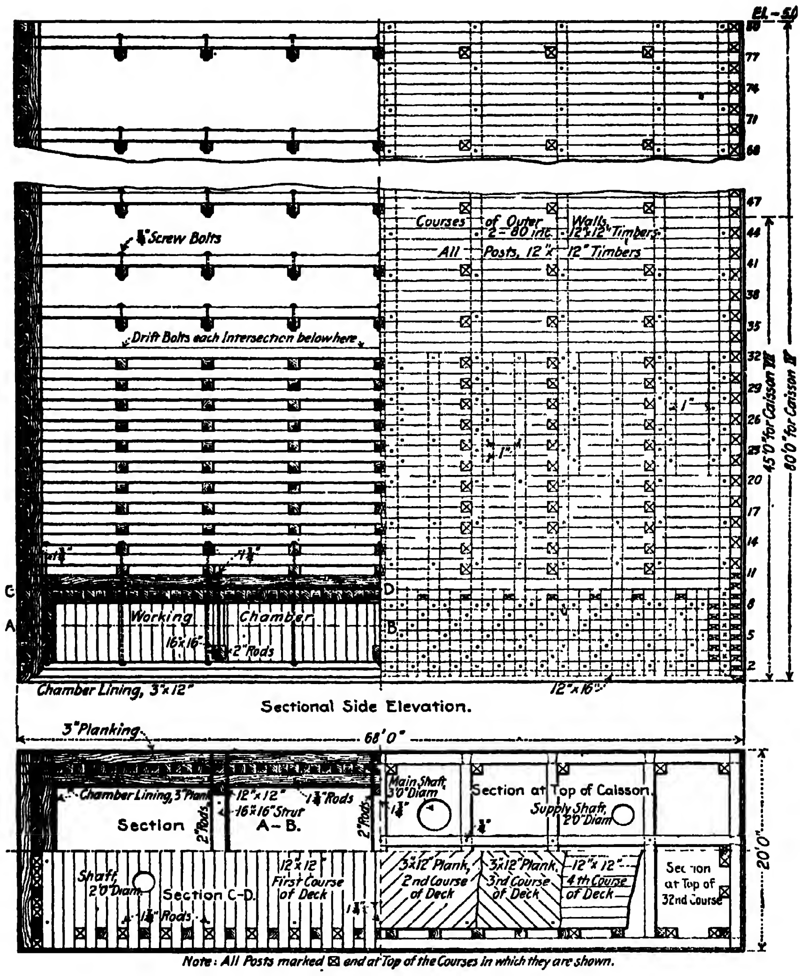


FIG. 96e.—Pneumatic Caisson for Broadway Bridge, Portland, Ore.

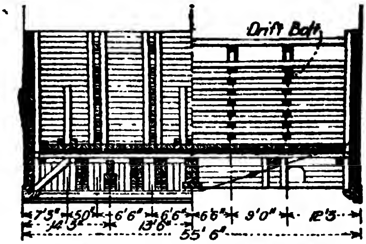


FIG. 96c.—Quebec Bridge Caisson.

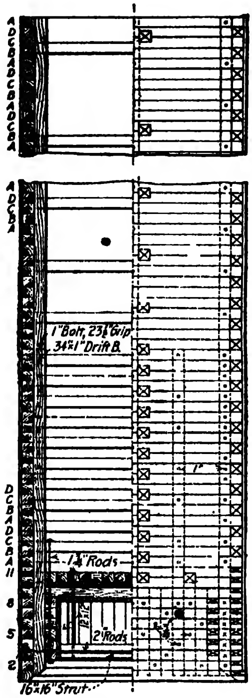


FIG. 96f.—Broadway Bridge.

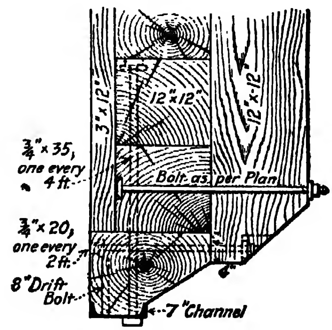


FIG. 96h.—Section of Cutting Edge, Broadway Bridge.

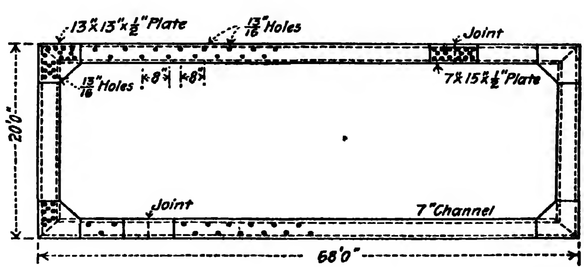


FIG. 96g.—Plan of Steel Cutting Edge, Broadway Bridge.

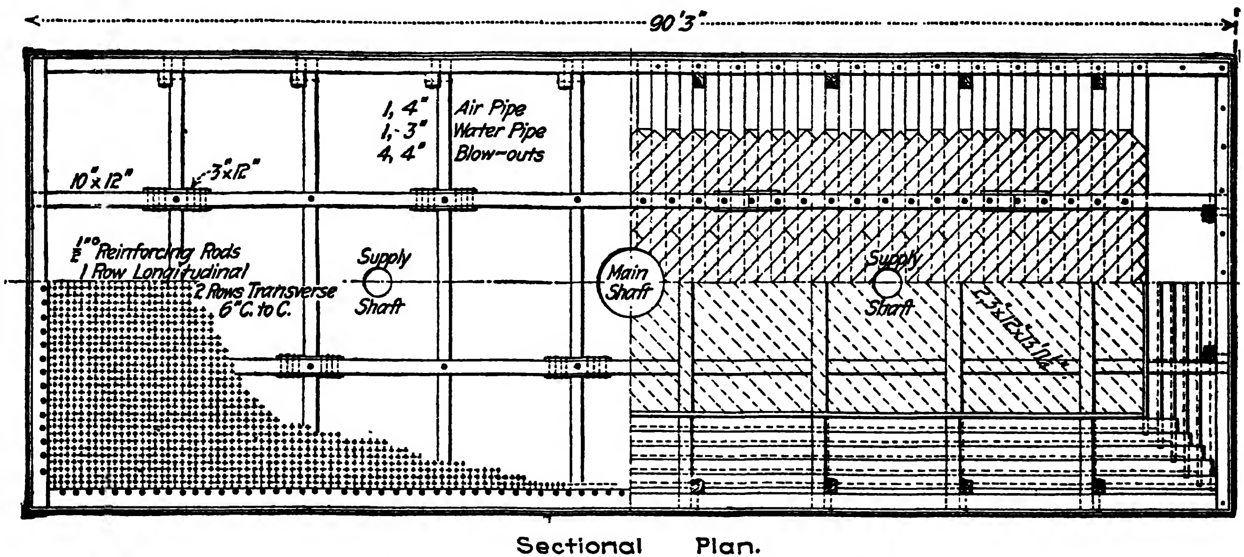


FIG. 96i.—Pneumatic Caissons for the River Piers of the Municipal Bridge at St. Louis, Mo., 1910.

The roof of the caissons for the Municipal bridge consisted of a single solid course of transverse timbers, sheathed on the upper and lower surface with 3- by 13-inch planks, placed diagonally and well calked. This roof served as a form for a layer of concrete placed on it and reinforced with two rows of transverse and one row of longitudinal 1-in. round rods, spaced 6 inches center to center, and placed a little above the upper layer of sheathing. Sinking the caisson was not commenced until this concrete had hardened.

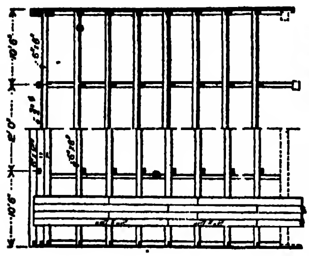


FIG. 96l.—Framing of Cofferdam, Municipal Bridge.

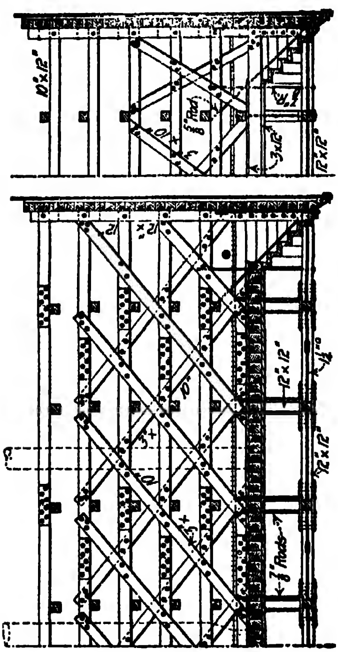


FIG. 96j.—Half Longitudinal and Half Transverse Sections, Municipal Bridge.

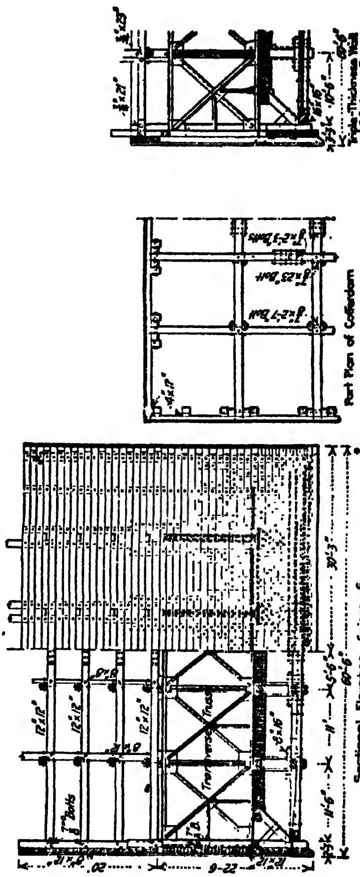


FIG. 96m.—Pneumatic Caisson for Metropolis Bridge, Metropolis, Ill.

The same form of roof was used in caisson 4 of the Broadway bridge, Portland, Ore., as shown in Figs. 96*e* and *f*.

REINFORCED-CONCRETE ROOFS.—Decreasing the amount of timber in the roof offers two advantages First it increases the strength of the pier foundation, the timber being the weakest element in the structure; and, second, it decreases the compressibility of the foundation. The cost is also greatly lessened.

The description of the roof construction for the Municipal bridge is given under FIG. 96*i*.

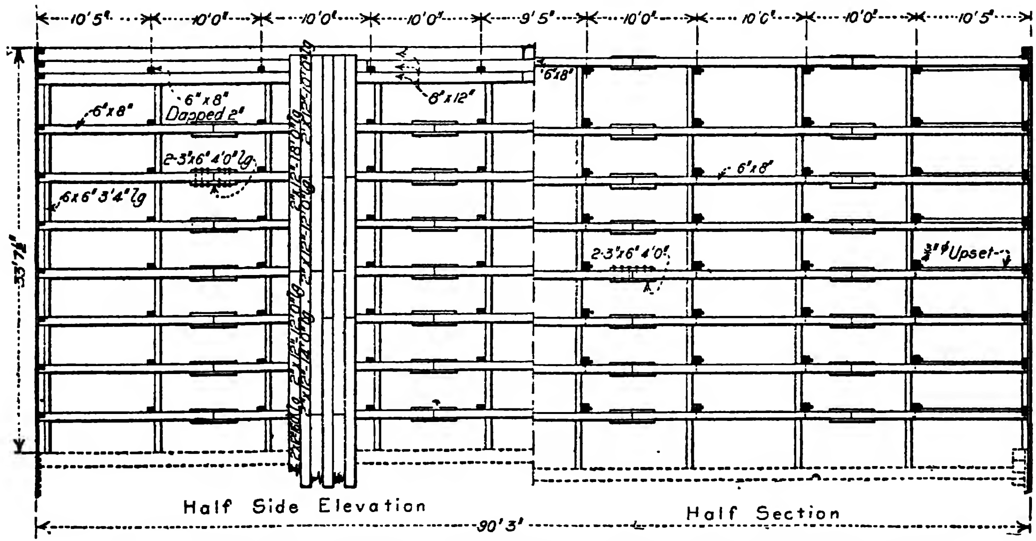


FIG. 96*k*.—Framing of Cofferdam on Top of Pneumatic Caisson and Crib, Municipal Bridge.

tongue-and-grooved planks. Here numerous bulkheads made a thin roof possible.

Figure 96*m* shows the 60½- by 110½-foot caisson used for the Metropolis bridge, where only a single thickness of timber was used for the roof. The concrete above was heavily reinforced, steel trusses taking the weight of this concrete until it had hardened.

ART. 97. SIDES OF WORKING CHAMBER

The sides of the caisson should be made strong and rigid enough not only to take the direct vertical loads, but also to withstand safely sudden lateral thrusts, eccentric loads due to unequal sinking of opposite sides, etc. To prevent leakage of air outward and of water inward all joints should be thoroughly calked. The necessary thickness of walls will depend somewhat on the clear height of the working chamber, as well as on the kind of material through which the caisson is to be sunk. The clear height should not, however, vary much from 6 feet.

The sides must be vertical. To batter the sides for the purpose of reducing the friction is to invite trouble. Such a design makes it more difficult to sink the caisson plumb, and is apt to increase instead of decrease the friction by allowing boulders to roll into the open space.

Practically all working-chamber sides are constructed of two forms: namely, that in which the vertical section is V-shaped, and composed of two walls; or that in which the vertical section is essentially a rectangle and composed of a single wall. The former has the advantage of being more rigid and so requires less bracing, while the latter has the advantage of permitting excavation under the cutting edge to be more easily made.

In the V-shaped form the space between the outer and inner walls may be built solid with timber, as was done in the east-abutment caisson of the St. Louis arch bridge; or it may be made hollow and afterward filled with concrete, as was done in most of the caissons designed by G. S. MORISON, a typical form of which is shown in Fig. 96*a*. Here the outer wall was

made of 12- by 12-inch timbers, sheathed on the outside with two layers of planking, the outer one running vertically and the inner one diagonally. The inner wall consisted of a single thickness of 17- by 17-inch timbers sheathed with 4-inch planks and tied to the outer wall with rods.

The St. Louis Municipal bridge caissons (Fig. 96*j*) had outside walls of 10- by 12-inch timbers, sheathed with two courses of planking: one 3- by 12-inch, running diagonally, and the other, 2- by 12-inch, running vertically, the latter being on the outside to reduce friction in sinking. The inner wall was formed of 4- by 12-inch horizontal planks, stepped and supported at intervals of 10 feet on vertical struts. The small size of material used in this wall was made possible by reinforcing the concrete in the space between the walls. Stepping the wall made it possible to count on the horizontal projection of this inner wall as taking load when the caisson was filled with concrete and in its final position. This cannot be done when the wall is on a slope. A further advantage is that the projections gave better control of sinking, there being less danger of sudden drops than when the wall is sloped.

The rectangular section of side wall is used more widely than the triangular, on account of the facility with which the spoil near the sides may be excavated. Figures 96*e* and *f* illustrate a good example of this type. It is composed of a double thickness of horizontal 12- by 12-inch timbers, separated by a single thickness of vertical 12- by 12-inch timbers, some of which extend up beyond the caisson to form a part of the crib. Both the outside and the inside faces of the wall are faced with 3- by 12-inch planks. Figures 96*b*, *c* and *d* also illustrate the same type.

The design of the walls of the Delaware River bridge caissons, described in Art. 103, represent a considerable advance in caisson-design practice as shown by the following table taken from an article by C. E. CHASE, published in the Journal of the Franklin Institute, November, 1923. This table gives a comparison of the strength of the working-chamber walls of four caissons.

Caisson	Outward thrust per foot, pounds	Inward thrust per foot, pounds
Second Quebec bridge	2,000	26,000
Manhattan bridge	5,000	36,000
Metropolis bridge	7,000	87,000
Delaware River bridge	55,000	70,000

ART. 98. DETAILS OF CUTTING EDGE

The cutting edge, as the part of the caisson which rests on the ground is called, must be designed to serve four functions: first, it must be sufficiently strong and tough to stand the strains and abrasive action of sinking; second, it must be of a form which will allow the caisson to sink readily without excavating under the cutting edge; third, it must have bearing surface enough to prevent sudden sinking when a soft stratum is encountered; and, fourth, it should be so designed that air cannot readily escape under the same. To fulfill the first requirement the cutting edge is usually made of some tough and strong wood, such as elm, or else is shod with a metal plate or piece of tough wood. The second and third are conflicting requirements; for the second a true knife edge is the ideal form, while for the third a considerable breadth of bearing is desirable. As constructed, the width will vary from about 4 inches to 18 inches. To meet the fourth requirement, a vertical plate extending about 6 inches below the cutting edge is often used. Where the soil is dense, this plate may be dispensed with.

Many engineers at present favor the blunt cutting edge in preference to the sharp one. T. K. THOMSON's experience is, that where the knife edge is needed, *i.e.*, in hard material, to allow getting close to the outside edge for excavating, it would cost too much to make the cutting edge strong enough, and where the material is soft a knife edge is not needed.

Figure 96*d* illustrates the use of a timber wearing plank on the cutting edge. It was 6 by 12 inches in section, the main timber forming the cutting edge being 30 by 30 inches in section, while

the upper inner corner of the latter was rebated 9 inches to form a seat for the feet of the vertical wall timbers. The advantage of a timber over a metal cutting edge is that less time is required to obtain it, and also it is much more easily replaced when broken.

The form of cutting edge used by G. S. MORISON, illustrated in Fig. 98*a*, consisted of a horizontal and vertical plate, the latter being stiffened at intervals and fastened to the horizontal plate by steel diaphragms, which are stiffened on the edges by three angles. The horizontal plate extended under, and was fastened to both the lower surface of the bottom timbers and the lower edge of the outside sheathing, while the vertical plate was fastened to the same outside sheathing. Near the bottom the vertical plate was reinforced with two others. Figure 96*a* shows the appearance of this cutting edge in place.

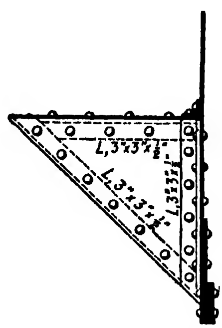


FIG. 98*a*.—Details of Cutting Edges.

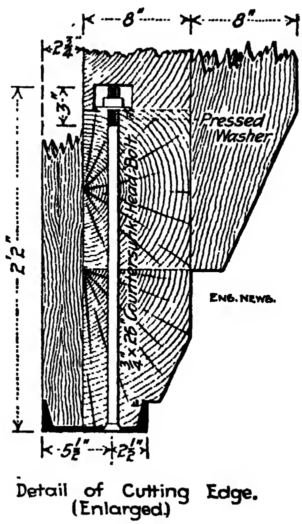


FIG. 98*b*.

The bottom timber of the caisson shown in Fig. 96*j* extended out beyond the timbers above to protect the lower edges of the outside sheathing, while it in turn was protected by steel plates on all sides but the top. This form of construction, having a vertical plate on the outside and a horizontal angle with its vertical leg down and fastened by rivets to the vertical plate, and with its horizontal leg fastened to the lower surface of the lower timber, is widely used, but is not economical.

The cutting edge of the caisson used in the Kinzie Street drawbridge, Chicago, was formed with an 8-inch channel iron laid horizontally with flanges turned up as shown in Fig. 98*b*. The same general form was used on the Broadway bridge caissons (Fig. 96*h*), the only difference being that in the latter case the cutting-edge timber extended out to protect the bottom of the sheathing, while in the former case the channel iron served this purpose. This form of metal cutting edge is the most economical and was designed in 1901 by T. K. THOMSON.

ART. 99. BRACING OF CAISSON

Every caisson requires more or less bracing; the larger and higher it is the more bracing will it require. This bracing may be in the form of struts and ties near the bottom, running horizontally the length and breadth of the caisson, or it may be in the form of bulkheads, or trusses. The latter two usually serve the added purpose of supporting the roof.

The bracing in the 33- by 90-foot caisson of the St. Louis Municipal bridge, shown in Figs. 96*i* and *j*, consisted of eight transverse and two longitudinal lines of horizontal 12- by 12-inch struts spaced about 10 feet apart, with 1¼-inch adjustable rods on both sides of each strut. The struts at their intersections were braced with vertical 12- by 12-inch timbers and pairs of ⅞-inch rods extending to the deck of the caisson. A similar form of bracing was employed in the Bellefontaine bridge caissons, as illustrated in Fig. 96*a*, as well as in the Broadway bridge caissons (Figs. 96*e* and *f*) and Metropolis bridge (Fig. 96*m*).

The south main pier caisson of the second Quebec bridge, 55 by 180 feet in plan, was divided by timber bulkheads, as shown in Figs. 96*b* and *c*, into 18 rectangular compartments approximately 19 by 25 feet in size. These longitudinal and transverse bulkheads were, respectively, 24 and 12 inches thick, except the lower course, which was 12 inches thicker. All extended from the ceiling to about the top of the cutting edge. Each transverse bulkhead was trussed by a pair of

adjustable diagonal rods, the ends of which took bearing in the end walls at roof level, through beveled washers; in the center they bore on steel plates, the latter in turn bearing on both longitudinal and transverse bulkheads. The end walls on each side of the longitudinal bulkhead were braced by a solid-web knee brace 12 inches thick, reaching from the cutting edge to the top of the first transverse bulkhead. Between bulkheads the sides were knee-braced to the roof by single and double 12- by 12-inch struts inclined at an angle of 45 degrees.

The bulkheads of the east-abutment caisson of the St. Louis arch bridge were of very massive construction, being made of eight horizontal courses of timber, the upper course having eight timbers in it, making a width of 10 feet, while the bottom course had three timbers, making a width of 3½ feet. The numbers varied in the horizontal courses between these two values in such a way as to give a V-shaped section of bulkhead. The height was 9 feet.

A longitudinal wooden truss was used to brace the 31- by 79-foot caisson of the Havre de Grace bridge. It was 6 feet deep, the upper and lower chords being composed of two pieces of 12- by 12-inch timbers. The web members, both vertical and diagonal, were composed of timber struts and diagonal rods, the latter extending through the first deck course of the caisson. Cross-braces were placed between the bottom chord of the truss and the side walls.

ART. 100. CRIB CONSTRUCTION

Some writers consider the crib as a part of the caisson, but since the crib may sometimes be dispensed with and the pier built directly on the caisson, it will avoid confusion by separating the two. A certain height of crib is often built as an integral part of the caisson to facilitate floating the structure into place. The purpose of the crib is two-fold: first, it serves as a form for the concrete; and, second, it serves temporarily as a cofferdam to keep out the water. If the masonry or concrete work is kept sufficiently in advance of the sinking, the crib may

sometimes be dispensed with, but this is seldom done because it brings too much weight on the caisson. The crib is a permanent part of the foundation and usually its walls are a continuation of the walls of the caisson, perhaps slightly modified. The crib is thoroughly braced with longitudinal and transverse timbers left permanently in place.

Although it is customary to fill the crib with concrete, yet under some circumstances this may not be done. In the substructure for pier 2 of the Memphis bridge, where the nature of the soil made it necessary that the load on the foundation bed be kept down to a minimum, the pockets near the walls in the crib were left empty, while for about 15 feet down from the top of the crib a solid-timber grillage was used, thus decreasing the weight of the structure very considerably.

The crib for the south main pier of the second Quebec bridge had a wall made of a single thickness of horizontal 12-by 12-inch timbers to a distance of 25 feet above the cutting edge of the caisson, braced by inside vertical 12- by 12-inch timbers, spaced as shown in Figs. 96*b* and *c*, the latter being extensions of certain of the vertical timbers forming the sides of the caisson. The outside was sheathed with the same material as used for the caisson. The walls were braced with horizontal longitudinal and transverse struts 24 inches apart vertically, up to a height of 25 feet above the cutting edge of the caisson, dividing the crib into 90 pockets approximately 10 feet square. A similar bracing course was placed 29 feet above the cutting edge of the caisson; above this point there was no bracing, it being replaced with a concrete retaining wall reaching to the top of the crib, built against the walls of the latter and battered on the interior face, increasing in thickness from the top down. This was placed early in order to allow it to harden before any stress was put upon it. The advantage of this retaining wall is that it made the upper part of the crib a solid monolithic mass of concrete.

The crib shown in Fig. 96*a* had the bracing carried to the top and was notable on account of the manner in which the bracing was tied together with vertical rods. Here the lower courses

of bracing helped to carry the roof loads; for this reason the part of the crib up to the top of the rods passing through the roof may be considered a part of the caisson.

The walls of the cribs for the St. Louis Municipal bridge piers consisted for the most part of one thickness of 10- by 12-inch timbers, sheathed on the outside with one layer of 3-inch diagonal and one layer of 2-inch vertical planks. The bracing consisted of vertical 12- by 12-inch timbers and of eight rows of horizontal transverse and two of horizontal longitudinal 10- by 12-inch timbers. As shown in Fig. 96*j*, a large amount of 3- by 10-inch diagonal bracing was also used, giving a truss-like action to the bracing and greatly strengthening it.

The crib construction of the Broadway bridge is shown in Figs. 96*e* and *f*; the details are so simple that no explanation is necessary.

ART. 101. COFFERDAM CONSTRUCTION

Both durability and appearance require that no part of the crib extend above low-water level; and, moreover, to keep the obstruction to the current as small as possible, the crib is stopped and the pier commenced at a considerable distance below low water. In some cases, where the current has a high velocity, the pier is started at or below the river bed, or the upper part of the crib is built with pointed ends. For these reasons, unless conditions are such that the pier construction can be kept well above water-level, a cofferdam in which to build the pier becomes necessary. Ordinarily, cofferdams may be dispensed with only when the construction is carried on at low-water stages or when the friction and resistance to sinking is large. As a general rule, it is desirable to keep the weight on the caisson as small as possible, as this affords better control of the sinking. Even when possible many engineers prefer not to start building the pier until the caisson is sunk to final position, for only at such a time can the masonry be started in the correct position. The walls of the cofferdam are usually made of lighter construction than those of the crib, but they are always thoroughly calked, and braced by struts running the length

and breadth of the structure. As the pier is built up, these braces are removed and the walls are braced against the pier. On the completion of the latter the cofferdam is removed, if not the whole structure, at least that part above low water.

Figures 96*k* and *l* illustrate the cofferdam used for one of the piers of the St. Louis Municipal bridge. The left dotted lines represent the top course of crib and the right dotted lines the top of struts. The cofferdam, which was 33 feet 7½ inches high, consisted of a frame of horizontal 6- by 8-inch and vertical 6- by 6-inch timbers, sheathed with 2- by 12-inch planks. It was braced with 6- by 8-inch struts, 4 feet apart vertically, and in rows about 10 feet apart horizontally.

The cofferdam used for the Brooklyn pier of the Manhattan bridge, New York, N. Y., was one of the highest that has ever been used in pneumatic-caisson work, being 44 feet high and about 75 by 144 feet in plan. It was built in three sections, the sides of the first two sections being made of 10- by 12-inch horizontal timbers laid close and supported by 12- by 18-inch verticals, spaced 12 feet apart. On the outside two layers of 3- by 12-inch sheathing were placed, the inner planking being horizontal and the outer vertical. The upper section differed from the others only in having 8- by 12-inch instead of 10- by 12-inch horizontals.

ART. 102. PNEUMATIC CAISSONS OF CONCRETE

Pneumatic caissons built entirely of concrete have been used to some extent in Europe, but in this country the nearest approach to the all-concrete pneumatic caisson are those for the Beaver bridge, described in Art. 93. As there explained, most of the sinking was done by the open-well method. With the exception of a few cases, like the one just noted, the tendency in this country has been to use wood, but at the same time to decrease the amount formerly used by reinforcing the lower part of the crib concrete, as was done in the St. Louis Municipal bridge caissons. A covering of timber offers three advantages: first, it avoids the necessity of waiting for the concrete

to harden before commencing sinking operations; second, it offers less resistance to sinking because of the reduced friction on the sides; and, third, it forms a protection in sinking for the concrete of the sides.

The pneumatic process was used during the final part of the sinking of the Beaver bridge caissons in order that the bottom might be thoroughly cleaned, as well as to permit laying the concrete filling in air. The caisson was changed from the open to the pneumatic type in the following manner: It was first freed of water down to a level which permitted the placing of horizontal wooden frames in each of the wells at an elevation of about 9 feet above the cutting edge. Concrete was then placed on these forms, filling the wells, the first 7 feet being allowed to harden for a week before placing the rest. At the center of each well a vertical shaft, 3 feet in diameter, was placed to form a means of communication between the working chamber and the outside.

ART. 103. PNEUMATIC CAISSONS OF METAL

The abundance of timber in America has limited the use of the metal type to relatively few cases, while in Europe it has been extensively used.

The river piers of the St. Louis arch bridge, the first structure in this country founded on large pneumatic caissons, rest on metal caissons. Two reasons may be given for this fact: first, there was considerable uncertainty as to the action of a timber roof when subjected to the horizontal thrust from the superstructure; and, second, timber had not been used in caisson construction to serve as a precedent.

The caisson for the east pier, which was hexagonal in plan, with overall dimensions of 60 by 82 feet, had walls of wrought-iron plates ¾ inch thick, braced with iron brackets extending from the bottom to the top, and spaced 2½ feet apart. The roof was formed of ½-inch iron plates riveted to the lower flanges of 13 parallel iron girders, spaced 5 feet 6 inches apart. It was also supported by two heavy bulkheads of oak timber, 7 feet high, in the air chamber. These strong supports for the

roof were necessary because the latter had to take the entire weight of a 100-foot height of stone masonry.

The walls of the caisson extended above the roof to form an inclosure, in which the masonry was laid. No monolithic concrete was used in this structure. For some distance up the masonry covered the entire cross-section of the crib, but above this it was stepped off, the space between the iron envelope and the masonry being braced with timbers and filled with sand. For the west pier caisson the iron envelope was carried up but

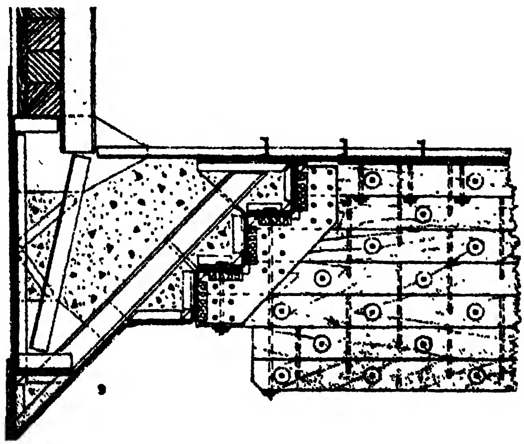


Fig. 103*a*.—Delaware River Bridge Caisson.

20 feet, after which the masonry was laid in the open, care being taken to keep the top of the same above water-level.

For large caissons the present tendency is toward the use of more steel. As noted in Art. 96, steel trusses were used to support the roof of the Metropolis bridge caissons, placed in 1914.

In making studies for the caissons of the Delaware River bridge (1922), designs were made involving the use of timber, concrete and a combination of steel and timber. The last was found to be the most economical and, as a consequence, the sides and roof of the working chamber and the supporting trusses above the roof were made of steel (Fig. 103*a*), while the bulkheads, crib walls and upper crib bracing were made of timber.

The metal pneumatic caissons for the Alexander III bridge, Paris, France, built in 1897, are among the largest of any type ever used. In plan, one caisson had the shape of a parallelogram (the angle being 84 degrees), the length of the sides being approximately 145 and 110 feet, transversely and parallel, respectively, to the axis of the bridge. The working chamber had a clear height of 6.23 feet and through this extended four transverse girders, each 6.23 feet high, their bottoms forming cutting edges, and dividing the chamber into five subchambers. On their upper flanges these girders supported 27 longitudinal girders, 5.2 feet deep, which carried the roof of the steel-plate platform that formed the deck of the caisson proper. The transverse girders had solid-plate webs for nearly one-third of their length at each end and open-web members in the central part. The longitudinals were ordinary latticed girders. The working chamber had a roof of steel plates 0.2 inch thick which were fastened to the lower flanges of the longitudinal, and to the upper flanges of the transverse girders. These plates did not extend horizontally through to the vertical sides of the caisson, but at the sides followed down the inclined end posts of the transverse girders, and at the ends followed the knee braces down to the cutting edge to give sloping inside walls on all four sides.

Between these inclined plates and the outer vertical walls was a triangular space filled with concrete. The outside wall plates and the transverse girders were all stiffened with knee braces extending from the cutting edge to the longitudinal girders.

The outside wall plates were reinforced on the lower edges by an outside vertical plate and the vertical flange of an inner angle, while the transverse girders were reinforced for bearing and cutting strains by adding two angles riveted, with their horizontal flanges upward, to the lower edge of the vertical web plate of the lower chord. The cofferdam above was 19.7 feet high and was composed of riveted and calked vertical plates, 0.118 inch thick, with a light angle-iron frame and light inclined angle-iron struts from near the upper edge and the middle of

the top of the transverse girders. The total distance sunk was 27 feet below ordinary water-level.[1]

ART. 104. CYLINDER PIER CAISSONS

The foundation for a cylinder pier is often placed by the pneumatic process, in which case, like the open-cylinder caisson, there is usually no particular point at which the caisson may be said to end and the pier begin. The pneumatic cylinder caisson is very similar to the open caisson in many cases, the only difference being that the former is fitted with horizontal diaphragm doors to form the air-lock. Often a part of the sinking is done by the open-caisson method and the remainder by the pneumatic method. As noted in Art. 95, the cylinder caisson was the first type of foundation to which the pneumatic process of sinking was applied in this country.

Figure 104*a* illustrates the cylinder piers and pneumatic cylinder caissons used for the Columbia River bridge at Trail, B. C. The shells were of steel plates from 5/16 to 7/16 inch thick. The lower 61 feet were formed of a double shell, the diameter of the inner shell being 3 feet, and that of the outer one 9 feet at the bottom and 6 feet at the top. Beginning at a point 8 feet above the bottom of the caisson, the inner shell was splayed out to meet the outer shell at the cutting edge, thus forming a working chamber 8 feet high. Near the bottom the two shells were braced together with diagonal lacing as shown in the diagram.

The upper parts of the cylinders were connected and braced by two vertical transverse 5/16- by 60-inch plates, 2 feet apart, braced together and the space between the two filled with concrete.

The air-lock was formed by placing two diaphragm doors in the inner shaft, one about 13 feet above the cutting edge and the other at a point 16 feet higher. As sinking proceeded, a third door, about 16 feet above the second door, was added,

[1] For further details the reader is referred either to the Engineering News, vol. 39, page 254, Apr. 21, 1898, or to the Engineering Record, vol. 37, page 275, Feb. 26, 1898.

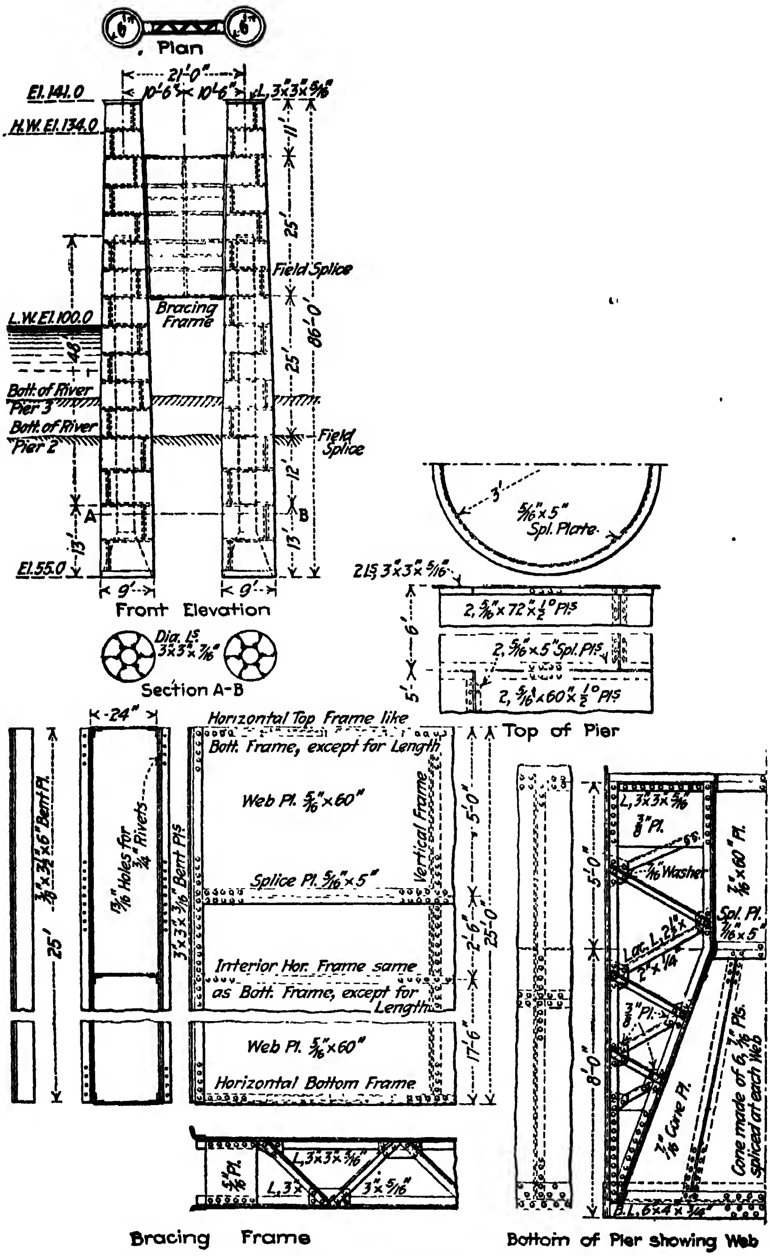


FIG. 104a.—Pneumatic Cylinder Caissons, Trail, B. C.

the upper two doors being used to form the lock, while the lower door was used for emergencies. These caissons were designed by WADDELL and HARRINGTON, and may be considered to represent current standard practice.

In the repairs of the Atchafalaya River bridge, each pier consisted of a pair of 8-foot diameter steel cylinders, filled with

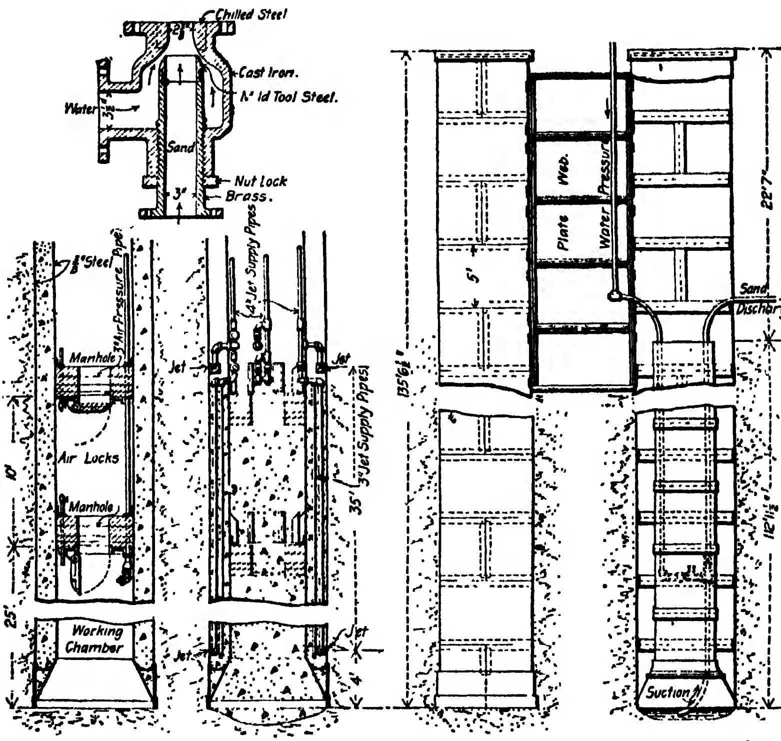


FIG. 104*b*.—Pneumatic Cylinder Caissons, Atchafalaya River Bridge.

concrete and braced together at the top by a stiffened web plate or diaphragm about 20 feet high, as shown in Fig. 104*b*. Each cylinder had, in addition to the outer 8-foot diameter shell, an inner concentric shell 5 feet in diameter, with a conical section uniting it with the cutting edge and closing the lower end of the annular space between the two shells. The shells were connected by four stiff webs. The inside shell terminated

about $22\frac{1}{2}$ feet below the top of the outer one, the latter having a total length of over $135\frac{1}{2}$ feet and was made with 5-foot rings erected in 10-foot sections. The working chamber was 25 feet high, and had a roof consisting of a 2-foot oak diaphragm made of four thicknesses of timber, with a circular hole 2 feet in diameter closed by a cast-iron door.

In the piers of the Glasgow bridge, which were sunk by the pneumatic process, the diameter of the outer shell was 15 feet, the thickness of the shell at the base being $\frac{1}{2}$ inch and at the top $\frac{5}{16}$ inch. The shaft, which was 3 feet 7 inches in diameter, formed the inner cylinder, and this was removed before filling the working chamber and air-shaft.

Almost no records exist of the use of the reinforced-concrete pneumatic-cylinder caisson. An example of this type, in which the first part of the sinking was done by the open-caisson method and the latter part by the pneumatic process, is given in Art. 105.

Art. 105. Combination Cylinder Caissons

With the cylinder caisson it is a simple matter to construct the cylinder to be used either as an open or as a pneumatic caisson. This makes it possible to utilize the advantages of both methods of sinking, the open caisson being used for that part of the sinking in which the material can be dredged or pumped out, and the pneumatic process for that part where boulders or compact material is met, and in finally preparing the foundation bed and placing the concrete filling in the working chamber.

The caissons for the Merrimac River bridge, between Salisbury and Newburyport, Mass., were of this type. Each caisson consisted of an 8-foot diameter cast-iron shell, the metal being $1\frac{1}{2}$ inches thick and cast in 8-foot sections. These sections had inside flanges bolted together and a mixture of red lead and linseed oil was paced between the joints.

The cylinders were sunk by inside dredging to a layer of boulders and gravel. They were then loaded with pig iron, air-locks placed on top and air pressure applied. No attempt was made to sink the caissons through the boulders, but instead

a novel method was used to transform this boulder and gravel layer into a good foundation bed. The pressure in the cylinder was reduced a little, allowing about a foot or more of water to rise. Portland cement was then mixed with the water to form a grout, which was kept well stirred while the air pressure was increased to force the grout into the gravel. On completion of the grouting, a depth of from 10 to 20 feet of 1-2-4 concrete was laid under air pressure, and allowed to harden, after which the remainder was laid in the open.

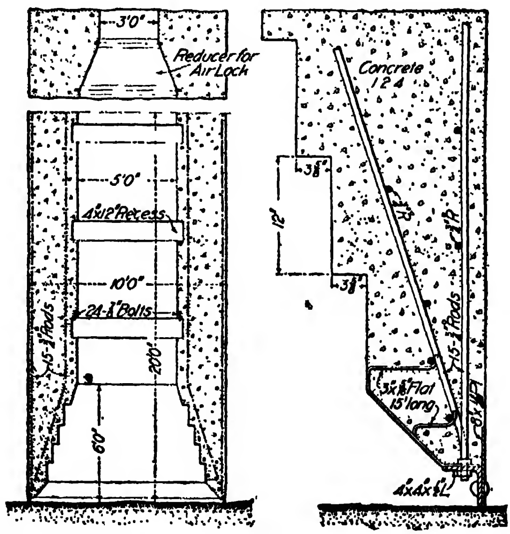


FIG. 105a.—Pneumatic Caissons of Reinforced Concrete for Bronx Viaduct of New York Connecting Railway.

Figure 105*a* shows the main details of concrete cylinder caissons used for foundations of the Bronx viaduct of the New York Connecting Railway. The caissons varied from 10 to 18 feet in diameter and were sunk to a maximum depth of 55 feet. The cutting edge was formed of a steel angle and steel plate, and the concrete composing the caisson was well reinforced with vertical and horizontal rods. When sinking through clay, the open dredging process was used, while in passing through quicksand, air-locks were placed in the upper part of the shafts and the pneumatic process used.

CHAPTER IX

PNEUMATIC CAISSONS FOR BRIDGES

ART. 106. SHAFTS AND AIR-LOCKS

The shafts, which form the means of communication between the working chamber and the outside, are circular in shape and in most cases are of steel plate $\frac{3}{8}$-inch thick; and in sections about 10 feet long, each section being flanged and bolted to the one above and below. Separate shafts are ordinarily used for men and materials, those for the men being about 3 feet in diameter, althought if an elevator is used they are often as large as 6 feet in diameter. The shafts for the removal of spoil are about 2 feet in diameter. Where the depths are only moderate it is customary to have a ladder built in the shaft used by the men, but when the depth is considerable a power elevator should always be employed, as it is extremely exhausting to climb a a long distance after working under high pressure. The men often use the excavating bucket as an elevator.

As explained in Art. 85, the air-lock is a chamber having two doors, one of which opens to the atmosphere and the other to the working chamber. These doors are so placed that the unusual air pressure will always force them against their seats, which have rubber gaskets to prevent the escape of air. The operation of the lock for men is as follows: The lower door being closed and the upper one open, a man enters; the upper door is then closed and compressed air slowly admitted to the lock, and as soon as the pressure in it becomes equal to that below, the lower door opens allowing the man to enter the working chamber.

The air-lock may be of any shape and of any desired size, the latter depending on the number of men or the amount of material it is desired to lock through at a time. The material lock is often but a section of the shaft.

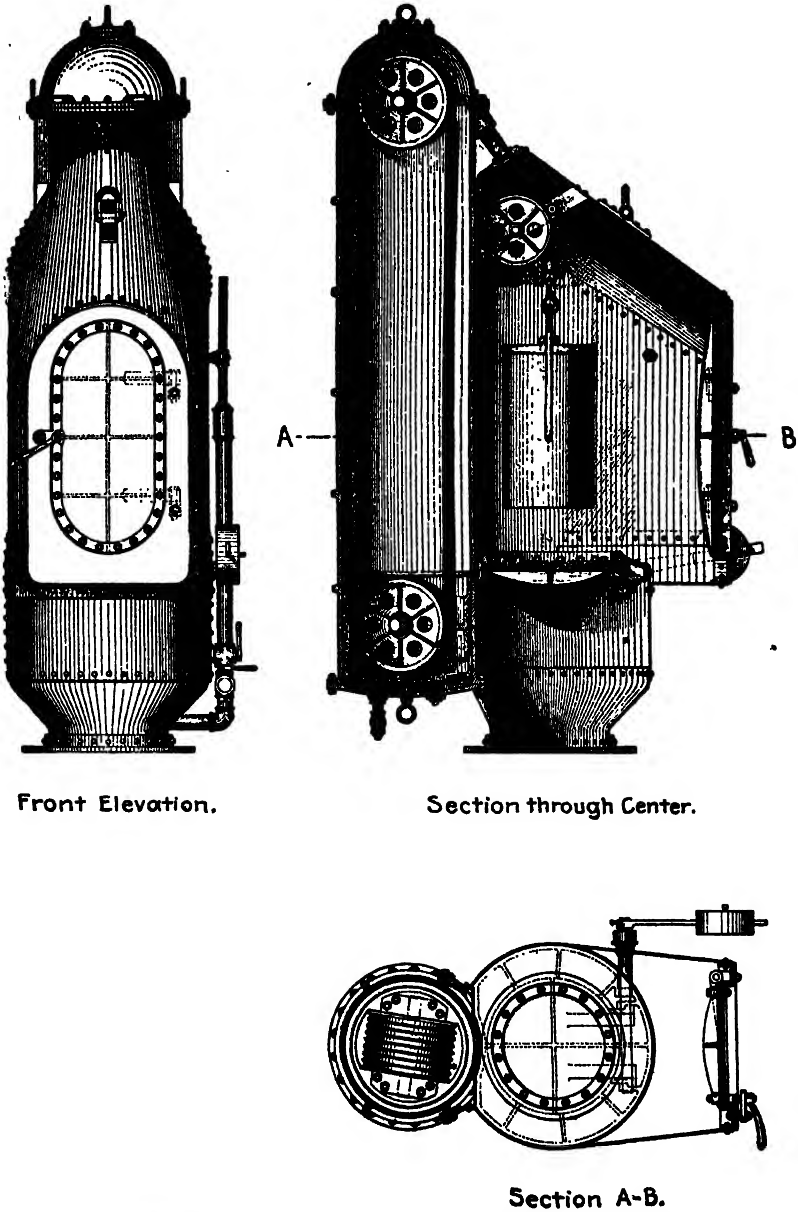


FIG. 106*a*.—Material Lock Used in Pneumatic Caissons of Memphis Bridge, 1891.

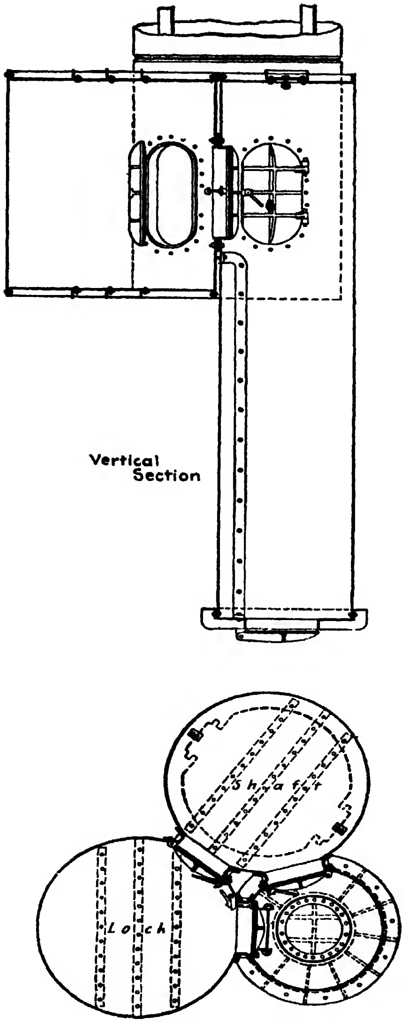


FIG. 106b.—Air-lock for Men, Memphis Bridge.

In the early caissons the lock was placed at the bottom of the shaft and extended down into the working chamber, but at present the material lock is always placed at the top of the shaft, while the man lock is placed either at the top or some distance up from the bottom. Caisson sinking with the lock at the bottom is a risky undertaking because a "blow-out," that is, a sudden outrush of air, will cause a like inrush of water accompanied by a rapid sinking of the caisson, which is almost sure to damage the lock. With the lock out of commission the men in the working chamber have no chance to escape, while if the lock is at the top the men can climb up and take refuge in the shaft above the level of the water. About the only disadvantage in having the lock on top of the shaft lies in the necessity of removing it each time a new section is added to the shaft; but with properly designed connections this can easily be done, and

without danger, by having an auxiliary door fitted to the lower end of the shaft in the roof of the working chamber which is closed when the lock is taken off.

Two forms of air-locks extensively employed for caissons used for the foundations of buildings are illustrated and described in Art. 122. The particular advantage which these

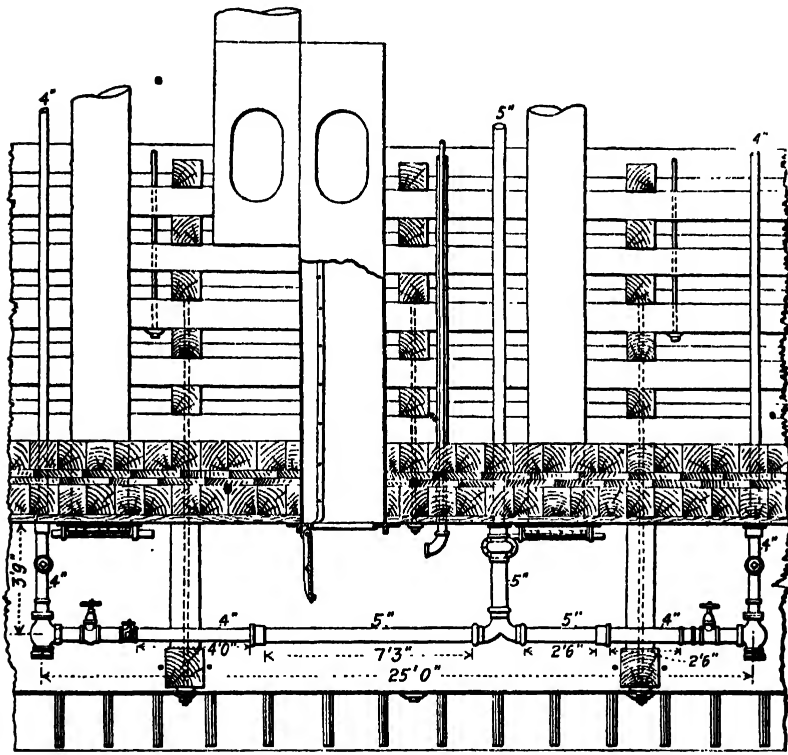


FIG. 106c.—Arrangement of Air-Lock, Shafts, Pipes, Etc., Bellefontaine Bridge.

types possess is that the bucket may be lowered into the air chamber, filled and taken out without detaching from the hoisting rope.

Another form of material lock which has been employed is illustrated in Fig. 106a, this particular one being used on the Memphis bridge caissons. The method of operation is described in Art. 110. The essential difference between this

and the types described in Art. 122 lies in the fact that here the upper door, instead of being in a horizontal plane, lies in a vertical plane at *B*. With this arrangement the material must be dumped out on being brought to the top, or else the bucket must be detached from the cable and taken out.

The form of lock for men employed on the above-mentioned bridge is illustrated in Fig. 106*b*. It is shown in position in Fig. 106*c*. [1]"The upper shaft through which the elevator-cage runs is a cylinder 6 feet in diameter, the air-lock itself is a cylinder 6 feet in diameter, and the shaft leading to the caisson, a cylinder 4 feet in diameter; the three cylinders are tangent to each other, and the shells are connected by cast-iron door frames carrying doors, while a fourth door opening outward was placed at the bottom of the lower shaft; in working, the door between the two shafts was always kept closed, and the door at the bottom of the bottom shaft was always left open; it was possible, however, if an emergency had arisen to use the lower section of the shaft as an air-lock in itself; when the filling of the working chamber was completed the bottom door was permanently closed."

ART. 107. DESIGN OF CAISSONS

It is impossible to compute even approximately the stresses in the various parts of a caisson and for this reason it is best largely to follow precedent. Engineers who are experts on caisson work have built many caissons, and by observing the weak points have developed strong structures with increasing economy. The examples given in the preceding articles are representative of the best forms in use, and are recommended to the careful consideration of engineers interested in this subject. For more extended information the reader is referred to the bibliography in Chap. XIX.

T. K. THOMSON, a consulting engineer who has specialized in pneumatic caissons, writes on their design as follows:

[2]"It is necessary to use considerable common sense and experience in attempting to calculate the stresses in a caisson.

[1] The Memphis Bridge, by GEO. S. MORISON.

[2] See "Construction," Nov., 1908.

As regards the deck, for example, it is very easy to calculate the weight to be carried by the deck and the stresses that would result therefrom, and we know that the air pressure acting up against the roof will counterbalance a great deal of this weight, making it, in fact, something like a pontoon floating in the water. But on the other hand, the air pressure is often slacked down to almost nothing in order to overcome the friction, and is raised again before much water has time to enter the working chamber; and sometimes an accident to the air plant will suddenly cut off the supply of air, throwing a tremendous stress on the roof. If the principal weight on the roof is concrete, it will in many cases be self-sustaining unless too fresh.

"The same with the sides. If the material were absolutely homogeneous all around and the caisson were sunk absolutely plumb, which almost never happens, and the air pressure were kept just equal to the outside pressure, then we would have practically no stress on the sides—but all practical caisson men have seen the sides of caissons collapse, and some very strongly built ones at that. A very much more frequent cause of accident than loss of air pressure is to strike some obstruction on one side, deflecting the cutting edge, and thus throwing much of the weight of the caisson on the weakened side, making bad worse . . .

"In building wooden caissons I very seldom halve the timbers or use dovetailed joints, preferring to use butt joints as much as possible with plenty of drift bolts. The trouble with butt joints, however, is that while a carpenter will make a dovetail or half-lap joint fit he will probably leave an inch or so play in a butt joint.

"The deck timbers, as well as those in the sides, should be planed on one side and one edge, for the sizes would otherwise vary too much to get a good job, while the planking for the outside and inside of the air chamber should be either tongue and groove, or the sides should be planed for a calking joint. The plank should, of course, have its faces also planed." Since very many drift bolts are required in fastening together the heavy timbers in wooden-caisson construction, it is desir-

able to adopt the proper diameter of holes to be bored. For the results of experiments on the holding power of drift bolts and the best ratio of the diameter of hole to that of bolt, see Art. 10 in JACOBY'S Structural Details.

ART. 108. BUILDING AND PLACING THE CAISSON

The caisson may be built on ways on the shore; on pontoons anchored near the shore, or over the site where it is to be sunk; or on a temporary platform supported by piles. Of the three methods, building on ways on the shore is the most widely used, but to make this method satisfactory the following conditions must obtain: first, there must be deep water near the shore; second, the soil must be sufficiently firm to hold the caisson, either with or without the use of bearing piles; third, there must be no danger of a high and rapid rise in the river; and fourth, the shore must not be at a great distance from the site of sinking.

Where satisfactory shore conditions do not obtain and where the water is deep and subject to sudden rises the pontoon method is the best. Where the depth of water is not great and where the river is not subject to considerable changes of level the method of using a temporary platform on piling is convenient. Caissons for abutments and buildings may usually be built directly on the ground near the site where they are to be sunk.

When built on ways the caisson sometimes has a false bottom fitted to it to reduce the depth of immersion, and a sufficient height of crib is constructed, preliminary to launching, to insure the top being well above water-level. After launching and towing to the site more crib is added, the false bottom removed and the caisson sunk to the river bed by placing concrete in the crib.

The launching ways used for the McKinley bridge over the Mississippi River at St. Louis, Mo., consisted of a number of rows of piles capped with timbers running at right angles to the river and on a slope of 1¾ inches per foot. Each caisson

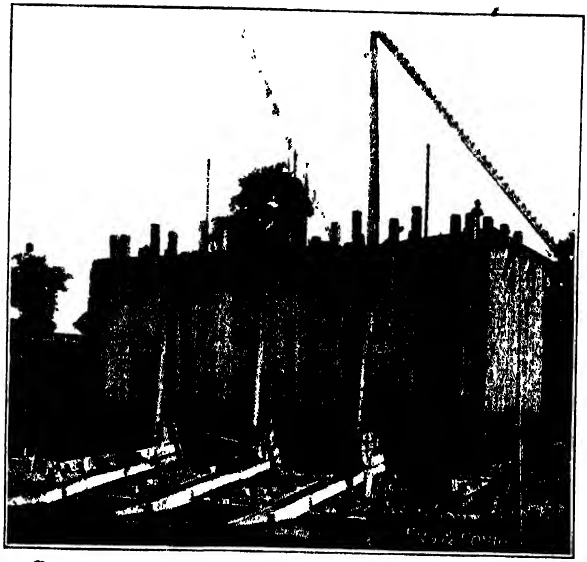


FIG. 108a.—Caisson on Launching Ways. Vancouver Bridge

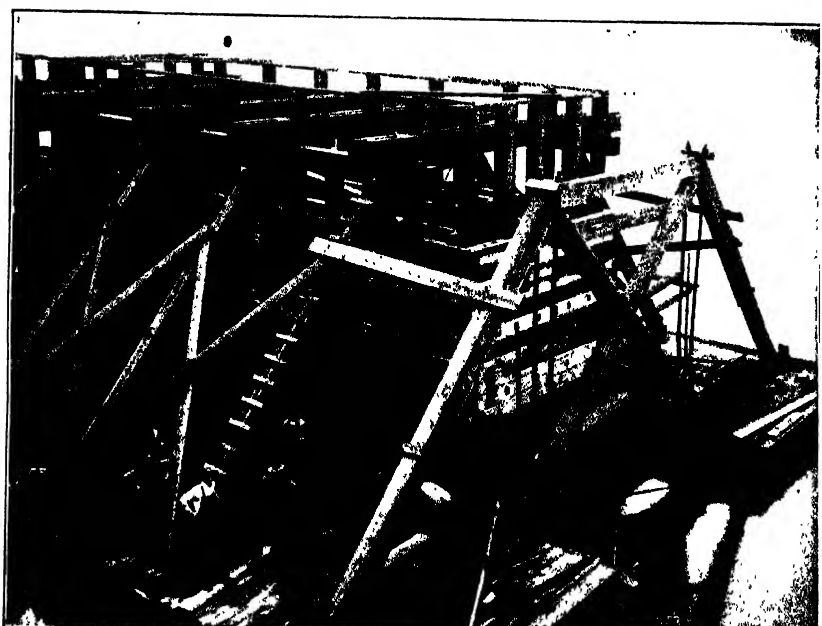


FIG. 108b.—Caisson Supported between Two Barges. Williamette River Bridge.

(Facing p. 342.)

FIG. 108e.—Launching a Caisson from the Pontoon in Which it Was Built. July 30, 1910. Municipal Bridge.

was built on shoes extending the full width of the caisson, the long side of the caisson being parallel to the river, and each shoe rested on a cap timber on which it slid during launching. These shoes were spaced about 6 feet apart and were so made that they projected down over the sides of the caps. They were bolted to the latter on the land side of the caisson. The caisson was built with its bottom in a horizontal position by using wedges between the caisson and the shoes. The launching was started by simultaneously sawing through the shoes below the bolts, which thus allowed the caisson to slide into the water.

Figure 108*a* shows the caisson for one of the piers of the Vancouver bridge, Vancouver, Wash., as it was being built on the

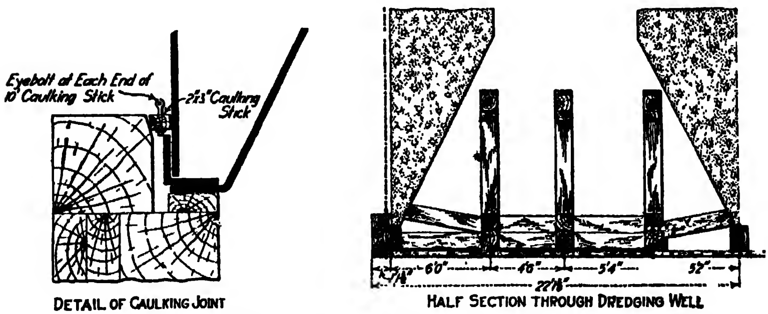


FIG. 108*c*.—False Bottom of New London Bridge Caisson.

launching ways. The general scheme was about the same as for the McKinley bridge caissons.

Where built on floats, either one or two pontoons may be used. Figure 108*b* shows one of the caissons of the Willamette River bridge of the Northern Pacific Railroad as it was being built between two barges or pontoons. The caisson was held between the barges until a height of 20 feet had been built up, when long screws were attached and the caisson lowered into the water. Two heavy trusses, one at each end, tied the barges together to prevent any unequal motion of the latter by the waves. Another caisson for the same bridge was erected on two pontoons, and after building to a sufficient height the

pontoons were scuttled by filling them with water, after which they were pulled out from under the caisson.

Figure 108*c* shows the false bottom of the New London bridge open caisson described in Art. 91. The steel cutting edges were assembled on the false bottom resting on ways, and the timber work was carried up about eight courses. Concrete was then placed in the cutting edges and, after hardening, the

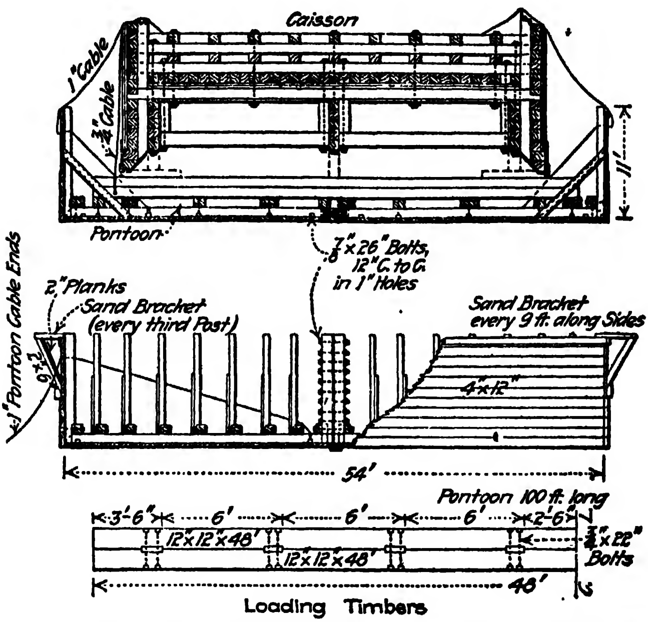


FIG. 108*d*.—Caisson-Launching Pontoon, New Memphis Bridge.

caisson was launched, towed to the site and further built up as high as possible and still permit the removal of the bottom. The dredging wells were then filled with water to river level by pulling out the calking stick and enough gravel dumped into the dredging wells to overcome the buoyancy of the false bottom.

The pontoon used in placing the caissons for the new Memphis bridge is shown in Fig. 108*d*.

The 78- by 144-foot caisson of the Manhattan bridge was built in a pontoon or float, 84 feet wide and 150 feet long, which

had vertical sides 8 feet high. The float was built of 3-inch planks bolted to vertical and horizontal timbers. It was built in two halves separated by a longitudinal joint along the center line. Blocking was set up on the floor timbers and on this the caisson was built, thus making the latter accessible from below. On completing the caisson, the joint between the two halves of the float was unlocked and sand dumped through the shafts of the caisson to the floor of the float to sink the halves of the latter, after which the same were pulled from beneath the caisson.

Figure 108e shows one part of the 40- by 100-foot pontoon of the St. Louis Municipal bridge caisson as it was being pulled from beneath the caisson. This pontoon, which was of the same type as that described above, was sunk by removing plugs from holes in the bottom of the pontoon.

In the construction of the Metropolis bridge caissons one end of the pontoon was so constructed that it would float out on a submersion of about 6 feet. Attached water boxes, containing about 75 tons of water when filled, were used to sink the pontoon clear of the caisson.

The caisson for the Passyunk Avenue bridge piers offer a good example of caissons built on a platform. Sixteen bearing piles were first driven in two longitudinal rows just clear of the caisson location. These were capped, and from these cap timbers four equidistant, transverse, 14- by 16-inch timbers were suspended by pairs of 1½-inch rods, 16 feet long, threaded the entire length, and each provided with two nuts. Each transverse timber was held by means of a steel saddle on the under side, against which the lower nut of the rod bore and the other nut took bearing on a washer on top of the pile cap. The transverse timbers were first screwed up tightly against the under side of the cap timbers and on these the caisson was built. After building the cribs to a height of about 26 feet the caisson and transverse timbers were gradually lowered by unscrewing the nuts from the rods, which permitted the caisson to float in its exact position.

ART. 109. SINKING THE CAISSON

If mud covers the river bottom this should be dredged out before placing the caisson, as it is cheaper to remove it in this manner than to excavate it within the working chamber. Great care must be exercised in grounding the caisson to place it in its correct position. If in tidal water, this may be done by placing concrete in the crib to an amount which will just ground the caisson at low tide. Then, by means of tackles attached to clusters of piles and to the caisson or crib, the structure is placed in its true position at high tide and grounded as the water-level lowers. Concrete is then poured into the crib to an amount which will prevent floating when the tide rises. Often, where the caisson is slightly out of position, it may be floated by admitting a small amount of air into the working chamber. As soon as enough concrete has been placed to put on air pressure safely to expel the water from the working chamber, men enter to commence sinking operations.

In clay, the excavation may usually be kept some distance below the cutting edge, which offers the advantage of allowing more head room for the men. This cannot be safely done in sand, as the water is very sensitive to changes of pressure and so it is not possible to raise the pressure very much from that corresponding to the head on the cutting edge. In one of the caissons of the Rulo bridge, a test well was sunk in clay 17 feet below the cutting edge without any increase in the air pressure, but when a 4-foot vein of gravel was struck the pressure had to be increased 8 to 10 pounds at once.

In sinking caissons the load is at first carried on the cutting edge, but as the caisson gradually sinks more of the load is resisted by friction on the sides and less by bearing on the cutting edge. Contrary to the usual custom, in the 55- by 180-foot caisson of the New Quebec bridge, the details of which are shown in Figs. 96*b*, *c* and *d*, and which for the most part was sunk through sand, the load was not at any time supported on the cutting edge.

[1]"Owing to the great size of the caisson, extraordinary precautions were considered necessary to provide against any unequal settlement, or any twisting or other movement of the caisson, which might tend to open up the joints and seams and consequently allow air to escape. On this account it was decided that the ordinary method of sinking, where all the load is carried on the cutting edge, would not allow the movements of the caisson to be sufficiently controlled during the actual sinking. The rather unusual method was therefore employed of carrying the entire load on the bulkheads and the roof, and no load at all on the cutting edge.

"The caisson was supported on 40 sand jacks, about 25 posts of 12- by 12-inch yellow pine, and 54 sets of blocking. The jacks and posts bore directly against the roof, while the blocking was piled under the bulkheads. When ready for a drop the blocking and posts were first removed by washing the sand from under them with a water-jet; then the whole caisson was lowered by operating all the sand jacks simultaneously. The sand jacks were of simple construction, each one consisting of a 29-inch steel cylinder closed at the bottom, having near the bottom two 3-inch holes with a sliding cover, and a plunger consisting of a single piece of timber fitting easily into the cylinder. The cylinder was filled two-thirds full of sand, the plunger inserted and its upper end blocked against the roof. The operation of lowering consisted in opening the lower holes and inserting a water-jet, thus washing out the sand.

"These jacks worked admirably, the result being that the caisson was sunk absolutely level and in its proper location. Before each drop a trench was excavated under the cutting edge to a depth of 2 or 3 feet, and filled with clay, which tended to prevent the escape of the air and also acted as a lubricant during sinking. This scheme was followed throughout the entire sinking and seemed materially to facilitate the operation."

Sinking the caisson is accomplished by excavating the material in the working chamber and by placing concrete in the

[1] Engineering News, vol. 68, pape 854, Nov. 7, 1912.

crib to weight the structure. The water-jet is sometimes employed to reduce friction on the sides.

ART. 110. REMOVING SPOIL FROM WORKING CHAMBER

Various devices have been developed for removing the spoil from the air chamber. Where the material is sand, the blow-out process or mud-and-sand pump is ordinarily employed; where clay is encountered, it is usually best to remove it with buckets, using some simple form of air-lock, or perhaps the clay may be mixed with water and the sand-and-mud-pump process used. Boulders must be removed through the air-locks.

BLOW-OUT PROCESS.—The blow-out process is a very simple affair, the principle consisting of using the pressure in the air chamber to drive out sand or mud when it is piled around the inlet of a pipe which leads from the working chamber to the open air. The diameter of the pipe is usually about 4 or 5 inches, the top being fitted with an elbow to throw the sand in a horizontal direction, while the lower part has attached to it a flexible hose of large diameter with a valve. To blow out the sand and mud it is only necessary to heap it up around the mouth, open the valve, and the material is then carried out with a high velocity; in fact, the velocity is so great that the pipe rapidly wears away. At the Havre de Grace bridge the elbow, which was of chilled iron, 4 inches thick, was worn through in two days. Many contractors use very hard manganese steel for these elbows. Considerable care must be exercised in placing the material against the inlet, for if a considerable amount of air is not admitted with the sand and mud, it will clog, while if there is too much air admitted it is a waste. It has been found advantageous to have small holes in the pipe above the inlet, as this gives more uniform action, tending to draw the material up instead of merely driving it and thus lessening the amount of air entering with the sand and mud.

In the construction of the Waverly bridge over the Missouri River a T-section was used instead of a goose-neck, the vertical

pipe extending up beyond the point of discharge. By capping the top of this pipe, the material discharged encountered an air cushion which eliminated the heavy wear at the bend.

The dry blow-out process is a very rapid and satisfactory means of removing spoil from the working chamber, although the consequent lowering of pressure in the air chamber causes a thick fog and also endangers entrance of water from the outside. If too much air is pumped into the working chamber a blow-out results, which is followed by a sudden inrush of water under the cutting edge. Care must be exercised to keep the pressure reasonably constant. The dry blow-out process is most satisfactory under fairly low pressures, although a head of at least 20 feet is necessary. This process was first used by Gen. WILLIAM SOOY SMITH in 1869 in building bridge piers in the Savannah River. Reaching a stratum so impermeable that the air pressure in the working chamber would not force out the water, a pipe was run down from above. On reaching a layer of sand it was found that the sand as well as the water was rapidly driven through the pipe.

SAND-AND-MUD PUMP.—The principle involved in this form of excavator is that of the induced current, where a quantity of water with a high velocity causes a reduction of pressure which draws the mud and sand—well mixed with water—into the pipe. Figure 104*b* illustrates the form often used. The water enters at the side under a high pressure and passes up through the small annular space, at which point, on account of the high velocity, the pressure is low. The lower part of the pump connects with a pipe or hose, the lower end of which rests in a pool of mud or sand and water. On account of the difference of pressure at the two ends of this pipe the mud is drawn into the pump and carried upward with the water, through a pipe which connects with the top of the pump. The essential difference between this form of excavator and the blow-out process is that in the former the water is the moving force doing the work, while in the latter it is the air from the working chamber. The water pressure used is ordinarily about 80 pounds per square inch. This method was first used by

JAMES B. EADS in the caissons of the St. Louis arch bridge. Figure 94*a* illustrates another form of the sand-and-mud pump.

In the Williamsburgh bridge, New York, the hose was extended to a sort of sump in the bottom of the excavation where its open end was placed below the surface of the water. Gravel, sand and mud were constantly fed into the nozzle by a laborer who raked it up and prevented clogging, and another man with a ¾-inch nozzle played a 50-pound water-jet against the soil to wash it into the sump.

For a description of this process as applied to open-caisson work the reader is referred to Art. 94. In some caisson work at Arran, Switzerland, instead of using a sump a horizontal hopper was employed, the discharge pipe leading from the lowest point in the hopper. A jet of water from a small pipe was constantly played on the material as it was fed into the hopper.

REMOVING MATERIAL WITH BUCKETS.—Clay is usually more cheaply removed with buckets than by any other method. Large rocks must be blasted to pieces and removed with buckets. As stated in Art. 106, where a form of lock similar to the Moran or O'Rourke lock is used, the bucket may be taken from the the lock without removing it from the hoisting rope. In the form shown in Fig. 106*a*, instead of running the hoisting rope to an engine on the outside, the hoisting is done by compressed air from the working chamber working in the cylinder shown on the left. In this cylinder runs a piston, the two sets of sheaves being so arranged that one stroke of the piston lifts the bucket the whole distance.

A novel device, called the water column, was used in the caissons of the Brooklyn bridge to remove the material. It consisted of an open shaft, the lower part extending into a sump which was kept full of water and the shaft itself was filled with water up to a point sufficient to balance the air pressure in the caisson. Workmen pushed the spoil under the shaft and from there it was removed by dredging with an orange-peel or clam-shell bucket.

The same device was used in 1921 in caisson work for a bridge across the Fuerte River on the Southern Pacific Railroad

in Mexico. Here the shaft was 5½ feet in diameter, extending 2½ feet below the cutting edge of the caisson.

ART. 111. CONCRETING THE AIR CHAMBER

When rock is reached, if the same is level, it is only necessary to clean off all loose material before depositing the concrete. On the other hand, if not level, some preliminary work must be done; if the rock has a uniform slope it should either be blasted down to a level surface or else stepped, unless very rough; although if the rock surface is at practically the same elevation all around the cutting edge of the caisson, but irregular within, little more than a thorough cleaning will be necessary. For those caissons founded on clay or hardpan, a level surface is easily obtained.

Caisson No. 10 of the Passyunk Avenue bridge landed on rock which had a slope of about 5 feet in the length of the caisson. As soon as rock on the high side was reached, the cutting edge on the low side was blocked with 6- by 12-inch timbers, 6 feet apart, after which excavation under the cutting edge was carried to rock and extended 1½ feet out beyond the cutting edge. This excavation was then filled with concrete.

In the caissons for the St. Louis Municipal bridge the rock surface was irregular, but no attempt was made to level it off or to bring the caissons to bearing throughout. Where depressions occurred the sand was removed and sacks of concrete were deposited on the rock and tamped under the cutting edge, after which concrete was placed in the working chamber in the usual manner.

In placing foundations for the 18½- by 87-foot pier of the new Thames River bridge at New London, Conn., because of the considerable slope of the rock in the upstream direction—approximately 35 feet in 90 feet—three 22-foot diameter caissons were used in place of one large one, the spacing being 35 feet center to center.

The concrete for filling the working chamber may be carried in through the material shafts and locks by means of buckets, or special arrangements may be made, by placing a cone-shaped

frame above the lower door, by which a yard or more of concrete may be dumped into the lock through the upper door. The latter is then closed and air admitted to the lock allowing the lower door to open and the mass of concrete to fall through the shaft to the working chamber. The conical frame prevents the concrete from remaining in the lock when the lower door is opened. For a description of the method used in placing the concrete in the working chamber, see Art. 191.

ART. 112. RATE OF SINKING

The rate of caisson sinking varies greatly, the larger the caisson and the harder the material sunk through the slower

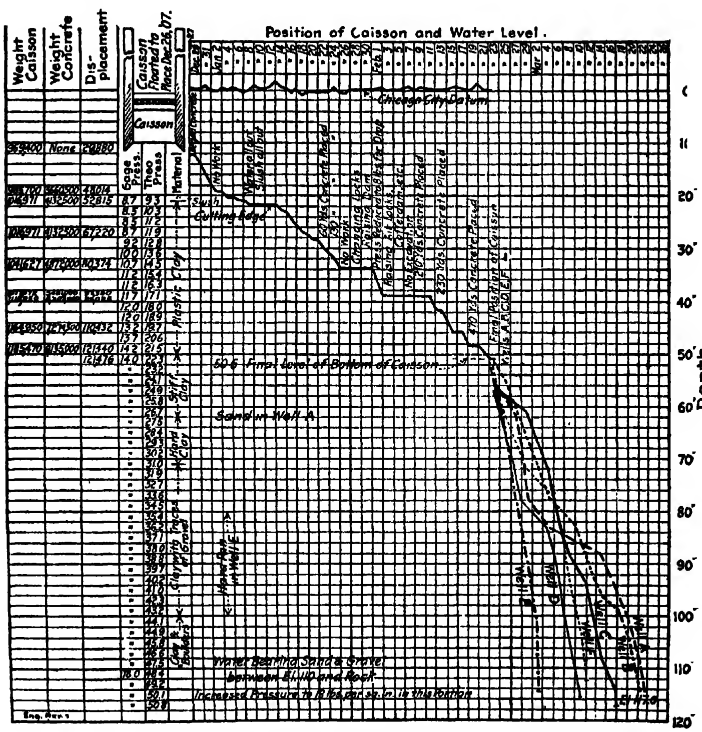


FIG. 112a.—Progress of Sinking Caissons, Kinzie Street Bridge, Chicago.

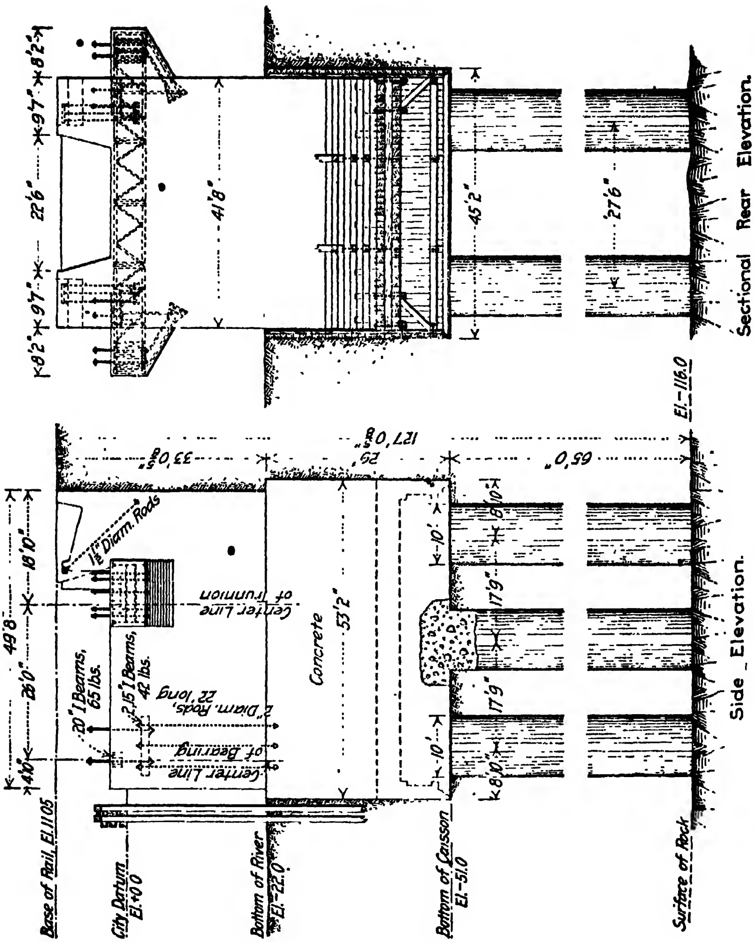
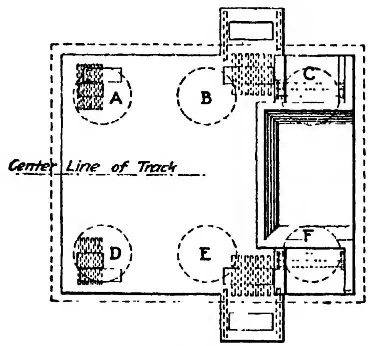


FIG. 112b.—Pneumatic Foundation for Canadian and North Western Railway Bridge, Kinzie Street, Chicago.

the rate. Sinking operations are usually carried on day and night, and the rate of sinking will vary from almost nothing where beds of boulders are encountered to as much as 3 feet a day where clean sand is met. Most engineers keep a chart of the progress of the work; Fig. 112*a*, which illustrates the progress in sinking one of the caissons of the Kinzie Street drawbridge, Chicago, is a very satisfactory form of chart to use. The caisson is shown in Figs. 112*b* and *c*. Instead of carrying the whole caisson to bed rock the cutting edge was stopped



Plan.

FIG. 112*c*.—(See also Fig. 112*b*.)

about halfway down and wells were then sunk the remainder of the distance.

In sinking pier D of the Memphis bridge, excluding long delays, an average rate of 1.5 feet per day of 24 hours was maintained through sand, and only 0.31 foot through clay, while for piers 2 and 5 of the Thebes bridge the average rates were 0.23 and 0.41 foot respectively; here hard gravel was encountered.

The rate per day of sinking the St. Louis Municipal bridge caissons varied from an average of 0.68 foot for pier 4 to 1.95 feet for pier 3, with 1.28 feet as an average for all caissons. The best progress in one day was 5.17 feet, while the best

seven-day run was 34 feet, or 4.86 feet per day. For the caissons of the McKinley bridge, St. Louis, the average rate for all caissons was 2 feet per day, with a maximum of 7.7 feet in one day.

ART. 113. FRICTIONAL RESISTANCE

Estimating the probable frictional resistance to be met with in sinking caissons is one of the most difficult features involved in the design. It depends upon numerous factors, such as the kind of material penetrated, the material composing the sides of caisson and crib, depth to which sunk, whether the sides of the caisson are vertical or flared, whether or not the water-jet is used and the amount of air leaking under the cutting edge.

In general, the frictional resistance per square foot of exposed surface of caisson and crib will seldom be less than 250 nor more than 800 pounds, although in boulder-strewn material it may be as much as 1000 pounds. Next to mud and silt, sandy soils offer the least resistance, especially when carrying large amounts of water, while clay will offer less resistance than material containing boulders. With uniform soil conditions, the unit friction will increase with the depth; for instance, at the McKinley bridge, which crosses the Mississippi River at St. Louis, the friction was found to be about 300 pounds per square foot of exposed surface at 40 feet, and 600 pounds at a penetration of 70 feet. In sinking 12-foot-square caissons through fairly soft, blue clay in the River Rouge in Detroit, the frictional resistance was 450 pounds per square foot of caisson surface.

Anything which tends to loosen the soil around the sides of the caisson and crib will decrease the friction, at least for a short time; escaping air has about the same effect as the water-jet in lubricating the material. Although flaring out the bottom of the caisson tends to reduce the side friction, yet, on account of possible wedging action by material falling into the open space above the bottom, and further, on account of the loss of guidance, pneumatic caissons are now practically all

made with vertical outside walls. Care should be taken to prevent the caisson from warping, for if the four sides are not in true planes the friction on the sides is greatly increased.

Table No. 113*a* gives values for the skin friction when the caissons were well down for a number of notable structures. Table No. 113*b*, taken from an aricle by H. L. WILEY in the Transactions of the American Society of Civil Engineers, vol. 62, page 113, March, 1909, gives values of friction for both open and pneumatic caissons.

In sinking the Commercial Cable Building caissons the frictional resistance varied from 250 to 300 pounds per square foot of exposed surface, while in the United Fire Insurance Co. caissons it was as high as 1000 pounds.

TABLE 113*a*.—SKIN FRICTION FOR PNEUMATIC CAISSONS OF BRIDGES
(Expressed in Pounds per Square Foot)

Name of bridge	Range for separate piers	Average	No. of piers	Materials penetrated in sinking caissons
Bellefontaine....	600–700	648	4	Fine sand, sand, coarse sand, boulders.
Blair Crossing...	330–410	381	4	Fine sand, coarse sand, clay.
Brooklyn........	600			
Cairo...........	622–932	750	10	Sand.
Havre de Grace..	308–489	400	4	Silt, sand, mud.
McKinley.......	600			
Memphis........	365–837	584	5	Sand, gravel, mud, clay, sediment, very tough clay, quicksand.
Miles Glacier....	620			
Nebraska City...	409–590	525	3	
New Omaha.....	472–673	617	5	Sand, gravel, some clay to bed rock.
Rulo...... ...	351–944	614	4	River sand, coarse sand, rubbish, clay, gravel.
Sioux City......	314–535	463	4	Fine sand, yellow sand, gravel, clay, boulders.
Williamsburg....	750			

General average for nine bridges, 554 pounds per square foot.

TABLE 113*b*

No.	Type of caisson	Method of sinking	Material penetrated	Skin friction	Depth below low water in feet	Area of base in square feet
1	Cast iron	Open excavation	Gravel, clay	240	60	125
2	Cast iron	Open excavation	Sand, clay	250	75	225
3	Cast iron	Open excavation	Sand	250	60	125
4	Wrought iron	Open excavation	Sand, clay	285	140	1000
5	Cast iron	Open excavation	Sand, clay, gravel	300	100	125
6	Cast iron	Open excavation	Sand	325	60	125
7	Cast iron	Open excavation	Silt	350	60	125
8	Steel construction	Open excavation	Silt, sand, clay	375	55	190
9	Cast iron	Open excavation	Silt, mud, clay	390	75	100
10	Timber construction	Open excavation	Sand	450	30	1300
11	Steel construction	Open excavation	Silt, clay	450	60	700
12	Steel construction	Open excavation	Silt, clay, sand	450	60	1200
13	Steel construction	Open excavation	Mud, sand	450	65	1300
14	Steel construction	Open excavation	Clay	450	75	1500
15	Iron construction	Open excavation	Sand, gravel, clay	480	65	200
16	Cast iron	Open excavation	Clay	500	60	125
17	Steel construction	Open excavation	Clay	700	65	1300
18	Masonry	Pneumatic	Sand, mud	205	40	75
19	Timber construction	Pneumatic	Clay	250	35	800
20	Steel construction	Pneumatic	Clay, sand	275	60	150
21	Timber construction	Pneumatic	Silt, sand, mud	310	75	2550
22	Timber construction	Pneumatic	Sand, clay, gravel	350	100	1200
23	Timber construction	Pneumatic	Sand, clay, boulders	400	48	1925
24	Timber construction	Pneumatic	Clay, sand, gravel	400	95	4500
25	Timber construction	Pneumatic	Sand, gravel, clay	425	55	1300
26	Steel construction	Pneumatic	Sand, boulders	450	68	2700
27	Timber	Pneumatic	Silt, clay, gravel	500	75	1800
28	Iron cyliner	Pneumatic	Sand, shale	525	60	1200
29	Timber construction	Pneumatic	Sand	540	75	1700
30	Timber construction	Pneumatic	Sand, clay	600	75	1400
31	Timber construction	Pneumatic	Sand, gravel, clay	650	80	2000
32	Timber construction	Pneumatic	Sand	650	90	1200
33	Timber construction	Pneumatic	Sand, boulders	660	101	2100
34	Timber construction	Pneumatic	Silt, sand, clay	900	45	1700

In testing the frictional resistance of a 4-foot diameter pier sunk 65 feet below ground level in Chicago by the Chicago method (Art. 128), the material under the pier was completely removed and on placing load on the pier it started to move when the force was 700 pounds per square foot of frictional area.

The highest value of frictional resistance was observed in 1910 while sinking the concrete caisson for the pivot pier of the reconstructed swing bridge of the Grand Trunk Railway at Black Rock Harbor on the Niagara River. The material pene-

trated was a very sticky red clay. The concrete open caisson weighed 8700 tons and 1084 tons of stone and pig iron were piled on top of it. The area was 10,235 square feet, thus giving a frictional resistance of 1912 pounds per square foot.

ART. 114. PHYSIOLOGICAL EFFECTS OF COMPRESSED AIR

The question of the physiological effects on the human system when working in compressed air is important from both humanitarian and financial standpoints. In the past almost all the important works employing compressed air have levied a heavy toll of suffering and death on the "sand-hogs," as caisson workers are commonly called. For instance, on the caisson work of the St. Louis bridge there were 119 cases of so-called caisson disease, with 14 deaths from the same, while on the Brooklyn bridge there were 110 cases of illness, with three deaths. Usually, no harmful effects are felt on entering the compressed air or while remaining in it, although occasionally ear drums are broken and blood vessels ruptured. Experienced men often find it impossible to enter when troubled with a bad cold. The chief trouble comes during decompression. The disease, which has been proved to be aeremia, may be divided into two classes: first, that in which the attack is light; and, second, that in which it is severe. The first form is characterized by very severe pains, chiefly in the joints, and closely resembles rheumatism in its effects. From the tendency to cause its victim to double up in agony it is commonly known as the "bends." When the attack is very severe it usually paralyzes its victim and is commonly fatal.

SENSATIONS FELT ON ENTERING THE AIR CHAMBER.—On entering the air-lock and having the air pressure turned on, some of the sensations felt are heat, slight giddiness and headache, pain in the ears, breathlessness, inability to whisper—caused by the resistance of the compressed air to the finer muscular movements of the tongue—and a feeling of resistance to movement owing to the density of the air. A slight discomfort is usually felt in maintaining equilibrium between the air pressure inside and outside the body, the most painful

being in the ears, as noted above. This may be overcome by closing the mouth and holding the nose, and at the same time trying to expel the air from the lungs; such action makes the pressure in the tympanic cavity equal to the outside pressure by means of the Eustachian tubes, which run from the back of the nasal passages to the cavity. This action should be repeated from time to time and as long as the pressure continues to increase. Relief may also be secured by swallowing. A cold makes the feat more difficult, since the Eustachian tubes are then somewhat blocked.

SENSATIONS FELT ON LEAVING AIR CHAMBER.—On leaving the air pressure the caissonier feels cold, and this is felt most keenly during the passage through the air-lock, being due to the expansion of the air in the lock, as well as to the expansion and liberation of gases in the body. To counteract the effects of this cold the air-lock should be warmed, the men should be given strong hot coffee to drink on emerging, and should dress warmly. Another characteristic of decompression is a dense fog, which occurs as the air becomes rarefied.[1] Another sensation often manifested on emerging is an itching, pricking feeling under the skin on all parts of the body; this disappears in a few minutes. The foregoing sensations are always felt; if the person is taken with caisson illness, the symptoms may be manifold.

[1] "The symptoms of caisson disease have been quite definitely established. First among these are neuralgic pains of an intermittent or paroxysmal character, and of varying severity. In the worst instances these pains, or cramps, as they are commonly called—although they are rarely accompanied by muscular spasms—are so intense as completely to unnerve strong men. This symptom is very seldom absent, and from it comes the popular name of 'bends' given to the disease. Another characteristic symptom which is always exhibited is a profuse cold perspiration. Another symptom which is of frequent occurrence, but which is not always exhibited, is pain at the pit of the stomach, usually, but not always, attended by vomiting. In about 50 percent of the cases observed, paralysis has been a characteristic symptom. The degree of paralysis varies from slightly impaired sensation or numbness in the extremities to complete loss of sensation and motion in the affected parts, which are

[1] Engineering News, vol. 46, page 157, Sept. 5, 1901.

most frequently the legs and lower part of the body. Finally, the sufferer usually exhibits a number of transient symptoms, which have their origin in the brain; these are headache, dizziness, double vision, incoherence of speech and sometimes unconsciousness. The duration of these symptoms varies from a few hours to several weeks in case of paralysis. In fatal cases congestion of the brain or spinal cord always exists. A very noticeable fact is that the attack of the disease never takes place while the subject is under air pressure, but always occurs while he is emerging from the compressed-air chamber or after he has emerged."

CAUSES OF CAISSON DISEASE.—Various theories have been advanced from time to time as to the cause of caisson disease. It is said that attention was first called to caisson disease at about the middle of the last century by TRIGER, who applied the use of compressed air in sinking some coal shafts at Chalons on the banks of the Loire. [1]"HOPPE SEYLER (1857) and THOMAS SCHWANN (1858), in Germany, and BUSQUOY (1861), in France, . . . gave the first correct suggestion as to the cause: *viz.*, that it was due to the setting free of bubbles of gas in the blood. Nitrogen gas is dissolved, according to the law of partial pressures, during exposure to the compressed air, and this dissolved gas, having no time to escape through the lungs, if the pressure be suddenly lowered, bubbles off just as carbonic acid escapes from aerated water when a bottle is uncorked."

In 1871, DR. JAMINET, the physician in charge of the compressed-air workers at the St. Louis bridge, became convinced from his studies that the disease was caused by too rapid a tissue change due to the absorption of an excess of oxygen.

About two years later, DR. A. H. SMITH, the surgeon in charge of the New York tower caisson of the Brooklyn bridge, arrived at the conclusion that the ill effects developed in working under compressed air were due to the pressure of the air forcing the blood from the surface of the body to the center and thereby causing internal congestion.

But it was PAUL BERT, who, by his remarkable experiments, published in 1878, proved the true cause of caisson disease to be the effervescence of gas in the blood and tissue juices. Since then such authorities as PHILLOPON, VON SCHROTTER, HELLER,

[1] Engineering Record, vol. 63, page 362, Apr. 1, 1911.

Mager, Haldane, Hill, Smith, Macleod, Greenwood and others have checked and extended Bert's experiments.

The gas which is present in the blood, and which comes out of solution if the pressure is too rapidly lowered, is mostly nitrogen, for if the chamber is properly ventilated there will be only a small amount of carbonic acid gas in the air, while the oxygen content dissolved by the blood is taken up chemically by the hemoglobin, as demonstrated by Dr. Haldane. As stated elsewhere, the tissue fluids, chiefly the blood, dissolve the air according to Dalton's law of solution of gases in fluids; *i.e.*, the amount of gas dissolved in a fluid is proportional to the pressure of the gas surrounding the fluid. Except for very high pressures, such as eight or ten atmospheres--values which will never attain in caisson work—these dissolved gases probably have no chemical effect on the system, and are quite harmless as long as they remain in solution. For high pressures the dissolved oxygen seems to have a toxic effect, causing a fatal inflammation of the lungs. Experiments have shown that with a pressure of ten atmospheres some animals will die in as short a time as 20 minutes.

When the pressure of the surrounding air is lowered, however, the dissolved gases, mostly nitrogen, are thrown out of solution in the form of bubbles. If the lowering of the pressure is done slowly, the gases are thrown out of the blood at the lungs without developing bubbles of any appreciable size. But if the pressure is rapidly lowered the gas bubbles stick, owing to their size, in the minute blood vessels and obstruct the flow of the blood, often causing the vessels to burst. The same condition obtains in the various tissues carrying juices saturated with gas; if these bubbles develop in the joints, the result is the "bends"; if in the spinal cord, paralysis; if in the heart, heart failure, etc.

Art. 115. Prevention of Caisson Disease

If the cause of caisson illness is a mechanical action due to the development of bubbles in the blood and fluid tissues, which in turn is due to too rapid decompression, then manifestly the

cure is decompression at a rate slow enough to avoid this phenomenon. The length of time will depend upon the amount of gas in the fluid tissues and upon the physical characteristics of the person being decompressed. The amount of gas in the fluid tissues will, in turn, depend upon (1) the degree of pressure in the working chamber and (2) the length of time under pressure. The length of time taken to saturate the body fluids at any particular pressure will vary greatly, depending upon the fatness of the subject, the amount of bodily work done, heat and moisture present, etc. From experiments DR. HALDANE concluded that in certain parts of the body where the circulation is rapid and the number of blood vessels high the tissue juices will become 50 percent saturated in 5 minutes, with complete saturation in 40 minutes; while other parts, lacking a copious supply of blood, will require 75 minutes for 50 percent saturation and about 4 hours for 90 percent saturation. Experiments show that the fatty tissues absorb about five times as much gas as does the blood and the rate of absorption is much slower; the rate of desaturation will be correspondingly slow. For this reason most authorities state that men inclined toward fatness should never be employed for compressed-air work. The better the circulation of the blood the more quickly and easily will the gases be thrown out of the system; for this reason only men in good physical condition should be employed. Old men, or those who have abused themselves by excessive drinking or other dissipation, should never be allowed in the working chamber.

Dr. EDWARD LEVY, after a study of nearly 2,000,000 decompressions accompanied by 4372 cases of caisson disease, thinks that normal lungs, normal kidneys and a good heart are the important things and that age and moderate fatness are probably not so detrimental as some believe.[1]

Authorities differ as to the time that should be allowed for decompression, but all agree that the usual time given is too short. Some urge a uniform rate of decompression, while others prefer stage decompression, that is, at first a rapid decompression to a certain pressure, followed by slower decompression.

[1] Technical Paper 285, U. S. Bureau of Mines.

Seldom is more than 15 or 18 minutes given to decompression; because the air-lock is small and as a consequence the men must maintain cramped positions in the same. Moreover, the lock is usually cold and filled with fog, due to the decreasing pressure. Properly, the lock should be large enough to allow the men some freedom of motion and it should be ventilated with warm, dry air. The French law, enacted in 1908, prescribes that for a head of water up to 65.6 feet not less than 21.2 cubic feet of air shall be provided for each man in the lock, and for depths above this not less than 24.7 cubic feet. During decompression the men should constantly move about and massage their various joints, as this has been found to assist materially in ridding the system of the gases.

MACLEOD suggests the following time for decompression as being safe:

Gage pressure	Length of shift	Decompression period
15 to 30	4 hours	½ to 1 hour
45 to 60	4 hours	1½ to 2 hours

In Germany, VON SCHROTTER, HELLER and MAGER, in 1900, published a work in which they laid down the principle that a uniform decompression at the rate of 2 minutes per 0.1 atmosphere, or 20 minutes per atmosphere, was safe.

The law of New York State (1925) governing the time of decompression for pneumatic caisson work for bridges and buildings is as follows:

Gage pressure in pounds.......	10	15	20	25	30	36	40	50
Time of decompression in minutes	1	2	5	10	12	15	20	25

It is specified that decompression must give a drop of half the maximum gage pressure at the rate of 5 pounds per minute, the remaining decompression being at a uniform rate.

The time of work in caissons, given by this law, is as follows:

Gage pressure	Times in hours					
	0–21	22–30	31–35	36–40	41–45	45–50
Time per day in caisson	8	6	4	3	2	1½
No. of shifts	2	2	2	2	2	2
Length of shift	....	3	2	1½	1	¾
Minimum time between shifts.	30 consecutive minutes	1	2	3	4	5

The theory upon which stage compression is based is that the gas in the blood will not effervesce until a marked diminution of pressure obtains, and as, to the point of effervescence, the gases are discharged at a rate varying with some function of the change of pressure, manifestly the more rapid the lowering of pressure the more quickly will the blood vessels be freed of the gases contained therein. Since almost no cases of aeremia are caused by rapid decompression from about 19 pounds gage pressure, it seems reasonable to assume that the pressure in the air-lock may be reduced that amount in about three minutes; from this point the pressure must be lowered quite slowly and should correspond to the natural rate of desaturation of the fluid tissues at that difference of pressure. When the gage pressure reaches about 19 pounds, the remainder of the decompression may be done quickly, for, as stated above, it appears that the average person can safely stand that difference of pressure. The fundamental idea upon which stage decompression is based is correct, but as there is but little experimental data and less precedent to guide one, it has not yet become general.

In sinking the pier caissons of the Delaware River bridge in 1922, there were 16.2 cases of "bends" per 10,000 decompressions for the Philadelphia side and 31.2 for the Camden side. The maximum depth of sinking below mean high water for the Philadelphia caisson was 58 feet and for the Camden caisson 82 feet

The time required for locking out from 15, 20, 30 and 35 pounds was 5, 10, 18 and 25 minutes, respectively. The working period at pressures less than 20 pounds was eight hours in

two shifts of four hours each, with an hour intermission between shifts; from 20 to 30 pounds, six hours in two shifts, with an intermission of three hours; and from 30 to 35 pounds, two shifts of two hours each, with a two-hour rest period between shifts. On this work there were no fatal cases of caisson disease.

In his study previously noted, Dr. Levy found that with men working under the New York regulations there was no caisson disease with pressures under 15 pounds. Up to a pressure of 29 pounds the number of cases increased at a fairly uniform rate, 248 cases per 10,000 decompressions being the rate at 29 pounds where there were two 3-hour shifts per day. At 30 pounds pressure and two 2-hour periods the rate decreased to 49 per 10,000 decompressions. The rate at 34 pounds pressure and two 2-hour shifts was 150 per 10,000 as compared with a rate of 77 where the pressure was 35 pounds and the daily working time two 1½-hour periods. This shows clearly the effect of the length of working periods.

Apart from the matter of slow decompression, other precautions, if taken, will do much to lessen the occurrence of caisson disease. Anything which tends to lower the vital resistance of the human system tends to promote caisson illness. For this reason the physical conditions under which the men work should be as good as it is possible to make them: There should be furnished plenty of fresh air; electric lighting rather than gas or candle lighting should always be employed, as the latter tends to vitiate the air; the air should be kept at as reasonable a temperature as possible, which means that it should be cooled during the summer time, as compression raises its temperature. At present this is done in practically all work, either by spraying the compressed air as it enters the working chamber, or else by passing it through a coil of pipes externally cooled.

[1]"It is well known that, in a confined atmosphere, man sooner or later suffers from the accumulation of poisonous gases. The criterion of this pollution of the atmosphere is the amount of a crbonic acid (CO_2) found present. Where the per-

[1] Cause, Treatment and Prevention of the Bends, by J. J. R. Macleod, Journal of the Associate Engineering Society, vol. 39, page 301, Nov., 1907.

centage of CO_2 in the air rises above 0.1 percent, evil effects are common. It should be clearly understood that these evil effects are not due to the carbonic acid itself, but to some other toxic property which the CO_2 content seems to run parallel with, and is, therefore, a measure of it. Now, under pressure, it is evident that such a gas will be still more dangerous. As a matter of fact, E. H. SNELL reports that an 'increase of CO_2 from 0.04 to 0.1 percent at 30 pounds pressure is the forerunner of much illness.' He found that by free ventilation of the caisson, so as to remove this CO_2, the illness dropped from seven cases a day to one case in two days . . . Ventilation is a matter which should be carefully provided for, since otherwise the CO_2 and other poisonous constituents of polluted air will have their usual depressing effects on the workmen and render them more prone to suffer from decompression symptoms."

Especially when sinking through foul material should care be exercised in keeping the air pure. T. K. THOMSON reports that when sinking through the foul bottom of the Harlem River the men suffered much from the bends, but when sinking through the clay below this, even though under a much greater pressure, very little trouble occurred. It is also noticed that a greater amount of sickness is apt to occur during concreting than at other times, this being due to the decrease in the leakage of the air, or inadequate ventilation.

CURE FOR CAISSON DISEASE.—The best and about the only cure for caisson disease is recompression with slow decompression. If the patient can be put into the air before the gas bubbles have had a chance to tear the blood vessels and fluid tissues a cure can usually be effected, but otherwise not. For this reason, a hospital air-lock, large and well ventilated, should always be maintained in readiness and the men should be housed near by, so that in case of delayed attacks they may be immediately recompressed.

This method of treatment probably dates from the time when E. W. MOIR, in 1890, introduced the medical lock for decompression in connection with the construction of the first tunnels under the North River, New York.

CHAPTER X

PNEUMATIC CAISSONS FOR BUILDINGS

ART. 116. GENERAL DEVELOPMENT

The application of the pneumatic caisson to building foundations has been restricted very largely to the tall buildings or "sky-scrapers" of New York City. Two conditions occur there which require this form of foundation: first, the necessity for carrying the column loads to bed rock; and second the presence of quicksand over the rock. Both the height of the buildings and the magnitude of the column loads make it imperative to found the piers on a very hard and unyielding stratum, preferably bed rock, since any irregular settlement is exceedingly dangerous and difficult to remedy in tall buildings. The presence of quicksand makes sinking to bed rock very difficult by other methods than that of the pneumatic caisson, due to the tendency of the material to flow into the excavation; while it is especially dangerous in the lower part of Manhattan Island, due to the liability of undermining adjacent building foundations, many of which rest on shallow foundations. The only disadvantage of the pneumatic method is its high cost, but this is fully justified where the security of very expensive buildings is at stake.

In its details, the caisson for a building does not differ materially, except in the matter of size, from the bridge caisson. It is customary in most cases to use separate piers for all the interior columns, these being circular or square in plan; but special conditions, such as the close spacing of two or more columns, or lack of lecarance, sometimes make it necessary to use one pier for two or more columns. Where the grade of the cellar floor is below the ground-water line, the wall piers often serve two functions: first, that of carrying the wall-column loads to rock;

and, second, that of acting as a dam or retaining wall to keep out the water. To accomplish the latter they must form a continuous wall, and hence they are made rectangular in plan, as wide as is necessary to give adequate working room and furnish the required stability as a dam or retaining wall—usually between 6 and 8 feet—and as long as can be conveniently handled, which is often as much as 30 feet. The ends of adjacent sections are then connected and made water-tight.

For the circular form of caisson the diameter may vary from about 6 feet as a minimum to 15 feet or more. For a rectangular section the largest that has ever been used for building foundations is in the New York Telephone Co. Building, where the largest caissons are 35 feet 3 inches by 38 feet 8 inches in plan. But more remarkable in many respects were some of the caissons used in the foundations of the Municipal Building, New York, one of which was 26 by 31 feet, and carried the load from five columns. In size this is not much larger than one used in the first building founded on pneumatic caissons, namely, the Manhattan Life Insurance Building, erected in 1893–94, where the caissons had dimensions of 21 feet 6 inches by 25 feet 6 inches. But in the magnitude of the single column loads and depth to which the caisson was sunk, a great development is apparent. The maximum column load in the Manhattan Life Building was about 400,000 pounds, while in the Municipal Building it was about 5,475,000 pounds; the depth of sinking below the street curb in the former was 54 feet, while in the latter it was 140 feet; the maximum air pressures (gage) used were, respectively, 15 and 48 pounds per square inch, the latter being within 2 pounds of the maximum allowed by state law. The 140-foot depth below the curb corresponded to a depth of about 112 feet below the level of general excavation. For the most part the depth to which pneumatic caissons for buildings have been sunk have ranged somewhere between 30 and 90 feet below the curb, the true depth of sinking and the hydrostatic head worked against being less than this, depending on the amount of general excavating done before sinking the caissons and the position of the ground-water level, respectively.

In the development of pneumatic caissons for building foundations a tendency was manifested early to do away with permanent shafts and roofs of the working chambers by making them removable. When present as a permanent part of the pier they tend to divide the pier into two separate monoliths of concrete, one an inverted T-shaped mass formed by the filling in the working chamber and shaft well and the other a ring-shaped mass surrounding the shaft. Removing the shaft before filling the well with concrete has now become standard practice, while the use of a temporary roof is very general. The two common methods of accomplishing the latter are: first, to fill the crib with concrete only after the caisson is sunk and the roof removed; and, second, to use a roof of reinforced concrete.

Pneumatic caissons made of wood, steel, wood and steel combined, and of reinforced concrete have been used. They are all satisfactory and the choice in any particular case will depend on current prices and the time required to obtain the materials and to construct the caisson. As a rule in this country, concrete is the most economical and steel the most expensive, and wood about halfway between.

ART. 117. CAISSONS OF TIMBER

Caissons made entirely of wood have been and are being extensively used, although not to the same extent that they are employed for bridge caissons. For square and rectangular caissons the construction is simple and, therefore, the time saved, as compared with those of steel or concrete, may be considerable.

Where the depth is not great nor the sinking difficult, the caisson and crib may be made of light construction. Such a form is exemplified in the caissons of the Rogers Building, New York, which varied in dimensions from 8 feet square to about 7 by 14 feet in plan, and were sunk to depths of from 28 to 60 feet below the curb, corresponding to about 35 to 40 feet below the excavation level. The sides were made of 3-inch

vertical plank fastened to two horizontal rectangular frames, one near the bottom and the other about 3 feet higher, and to the roof timbers, by two ⅝-inch bolts at every intersection, the bolt heads being countersunk into the planks on the outside. All joints were calked on the outside with oakum. The cutting edge was made by beveling the inside lower ends of the planking to a thickness of 1 inch. The lower horizontal, which was of 4- by 12-inch material, was but a few inches above the cutting edge and hence reinforced the same. The working chamber had a clear height of 6 feet and was covered with a roof made of two solid courses of crossed 8- by 8-inch timber, the lower course resting on 6- by 8-inch timbers bolted to the sheathing.

The sheathing projected 6 feet above the lower side of the roof of the working chamber and formed one section of the crib. The other sections were made in 14-foot lengths and were of the same general construction as the caisson sides. At the joints between the successive sections of the crib the ends of the sheathing were cut square and braced by a inside 6- by 12-inch frame, the latter being bolted to both sections to serve as a connecting flange. The cribs were braced at intermediate points by 6- by 8-inch horizontal timber frames.

A much stronger form of wooden pneumatic caisson is illustrated in Fig. 117*a*, which is one of the 12- by 24-foot caissons used in the Gillender Building foundations. The sides of the working chamber were composed of two thicknesses of 12- by 12-inch timbers sheathed on the outside and inside with 3⅞-inch material. The cutting-edge timber extended out beyond the walls, the outer part of the upper side abutting against the bottom of the outside sheathing, while the outside and bottom faces were protected by the cutting edge, which consisted of a steel angle and a vertical steel plate.

The roof consisted of three thicknesses of 12- by 12-inch timbers, the upper and lower ones running transversely and the intermediate one longitudinally. The under side of the roof was sheathed with 3⅞-inch material. The crib was composed of 3⅞-inch sheathing, braced at intervals by horizontal frames of 8- by 12-inch timbers.

If less strength is desired the walls may be made of a single thickness of horizontal timbers laid closely and sheathed on the outside and inside. This was done in the caissons for the Mercantile Building, the timbers being 6 by 10 inches, the latter dimension horizontal, while the sheathing consisted of 3- by 12-

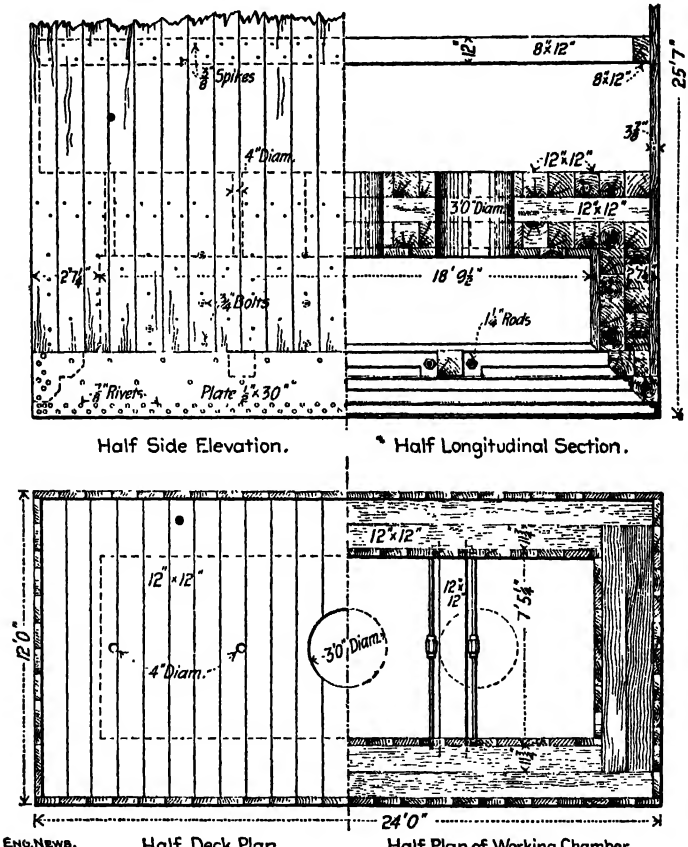


FIG. 117a.—Pneumatic Caisson of Timber Construction.

inch planks. In this case the roof was composed of a double thickness of 12-inch timbers.

The foregoing examples have permanent roof construction. The first wooden caissons doing away with this feature were those of the United States Express Co. Building. Here the wall

caissons were built with a width of 5½ feet, a height of 6 feet and lengths varying from 25 to 34 feet. The walls consisted of a single thickness of timber varying from 6 by 12 inches to 10 by 12 inches. Across the 10- by 10-inch top course were placed 3- by 3- by ¼-inch angles running transversely and spaced 3 feet apart, the vertical flanges being turned up. On these was placed a layer of 1⅞-inch tongue-and-grooved boards which served as a form for concrete placed on top of the same.

Instead of a crib, molds built of vertical tongue-and-grooved boards in sections 8 feet high, having the same length and

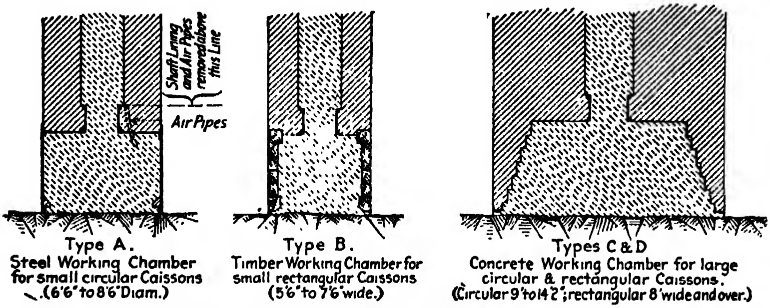


FIG. 117*b*.—Types of Working Chambers, Municipal Building, New York.

breadth as the caisson, were built on top of the latter to receive the concrete. They were held together by outside horizontal yokes, each made with 4- by 3- by ¼-inch angles forming a rectangular frame. Three yokes were used to a section, one at the top, one at the middle and one at the bottom. On completion of the forms, a 6-inch layer of concrete was placed on the roof forms; as soon as this had hardened somewhat, 2 feet more of concrete was added, and this 2½-foot thickness of concrete served as the permanent roof, the temporary panels underneath being taken off.

Figure 107*b*, type B, shows the type used in the Municipal Building for rectangular caissons 5½ to 7½ feet wide. It closely resembles those described in the two preceding paragraphs, the walls being made of a single thickness of 12- by 12-inch and 8- by 12-inch timbers, the bottom timbers being faced with 4- by 4-inch steel angles to form the cutting edge.

It was provided with a temporary deck of 2-inch planks notched into the walls and this deck was removed after the first layer of roof concrete had hardened.

The approximately square caissons used in the foundations of the Adams Express Building, New York, are illustrated in Fig. 117c. The working chamber was 6 feet high and had sides com-

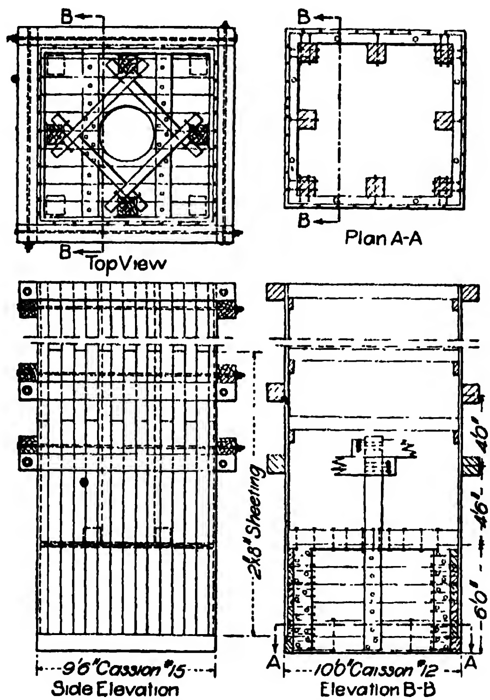


FIG. 117c.—Caissons, Adams Express Building, New York.

posed of 4- by 12-inch timbers dressed on all sides and calked. These sides were braced with vertical inside 12- by 12-inch timbers at the four corners and at the midlengths, the latter extending beyond the caisson. The outside was sheathed with 2- by 8-inch vertical tongue-and-grooved planks, which extended up beyond the caisson to serve as a form for the concrete above the caisson. This sheathing took bearing against horizontal 12- by 12-inch timbers spaced 4 feet apart vertically,

and arranged in pairs at right angles to each other, each pair being connected together with screw-ended rods. The sides were held apart by 3- by 8-inch struts, which were removed when the concrete had been placed.

The cutting-edge timber was 6 by 12 inches in section, beveled on the inner corner and projected beyond the horizontal timbers to cover the lower ends of the outside sheathing. The top of the working chamber was covered with 4- by 12-inch horizontal boards to serve as a form for the concrete above.

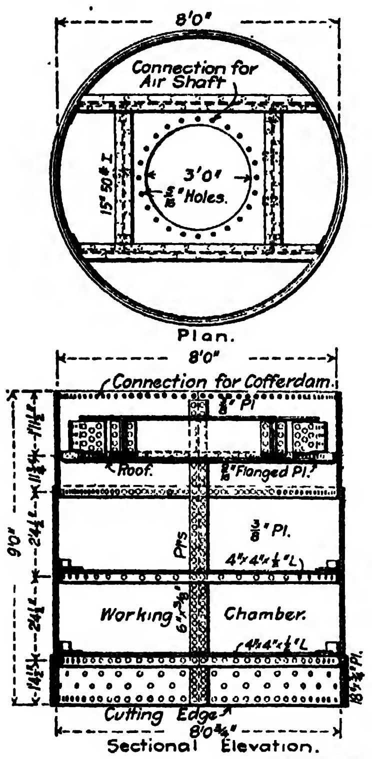


FIG. 118a.—Caisson for Inside Column. Foundation, Mutual Life Building, New York.

A 3-foot layer of 1-2-4 concrete was first placed on the deck, allowed to harden for 24 hours, after which a 6-foot layer was added every 24 hours. The deck sheathing was removed 48 hours after the first layer of concrete was placed.

ART. 118. CAISSONS WITH METAL SHELLS

The use of steel shells for small circular pneumatic caissons has become standard practice, but their use for caissons of a square or rectangular shape is rapidly decreasing. The advantages of the steel shell may be summarized as follows: first, as the thickness of the shell is small, there is a maximum amount of working space in the air chamber, as well as a maximum amount of space to be filled with concrete; second, for the cylindrical form it compares favorably in ease of construction with wood and concrete; and, third, it is easily made water-tight.

The first pneumatic caissons used for a building, those of the Manhattan Life Building, were made of steel, and were both circular and rectangular in section. Figures 118*a* and *b* show the details of both forms of caissons used in the foundations of the Mutual Life Building. The caissons were sunk to solid

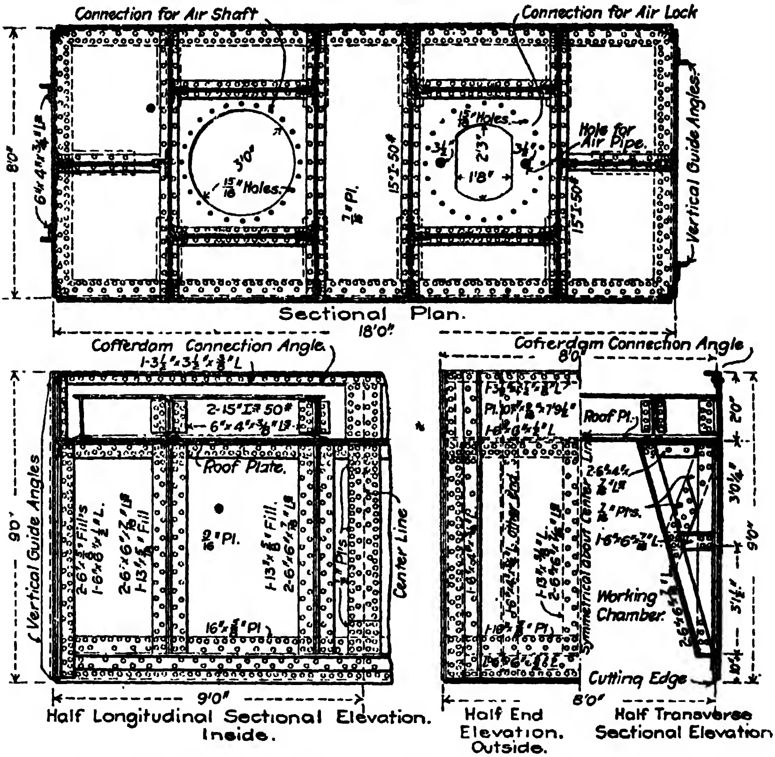


FIG. 118*b*.—Caisson for Wall Column, Mutual Life Building, New York.

rock, from 70 to 90 feet below the curb and from 50 to 70 feet below ground-water level. The roof of the cylindrical caisson was made of 9/16-inch steel plates riveted to the lower flanges of 15-inch I-beams, as well as to the shell of the caisson. The latter consisted of 3/8-inch steel plates braced at intervals with circular 4- by 4- by 1/2-inch steel angles. The lower part of the shell, reinforced with an 18- by 3/4-inch plate, formed the cutting edge. In the rectangular caisson the roof construction

was very similar to that described above; but the sides were braced with brackets, extending from near the cutting edge to the roof as shown in the illustrations.

The 3½- by 3½- by ⅜-inch angle riveted around the upper edge of the rectangular caisson formed a flange for the connection to the crib, which was built in sections 5 feet high, of ⅜-inch plate, with 3½- by 3½-inch angles for both top and bottom flange connections. Vertical 6- by 6-inch connection angles were used at the corners, and vertical 3½- by 3½-inch angles in the middle acted as stiffeners. For the circular caissons, butt instead of flange joints were used; here also ⅜-inch plates in 6¼-foot courses were employed for the crib. Both vertical and horizontal splices were butt-jointed with single splice plates on the outside.

Figure 117*b*, type A, shows the form of caisson used in the foundations of the Municipal Building, for all circular caissons having diameters between 6½ and 8½ feet. They were used in order to secure the maximum economy of space and material in the limited area of the working chamber. The height of the working chamber was 6½ feet, the walls being of 7/16- and ½-inch plates, shod with a 6- by ⅜- or 4- by ½-inch Z-bar cutting edge filled with concrete. The deck was formed of ⅜-inch steel plates slightly domed, was flange-bolted to the walls and air-shaft, and removed after the concrete above it—which was placed before sinking commenced—had hardened, this being the same method as that already described for wooden caissons. The section of the air-shaft shown remained a permanent part of the structure. The removable forms for the pier above the caisson were made of circular steel plate, held in place and reinforced with angle-iron rings, all of which are clearly shown in Fig. 118*c*.

ART. 119. CAISSONS OF WOOD AND STEEL

The caisson of wood and steel combined possesses some of the advantages of both steel and wood, but its origin is due largely to the fact that it was necessary to make a rush job in the pier sinking of a certain building, and for this reason only

FIG. 118c.—Sinking and Concreting Caissons, with Steel Forms. Municipal Building.

(Facing p. 376.)

those shapes of structural steel were used that could readily be obtained in the open market; the rest of the structure was made of wood, and the two materials combined in the simplest possible manner.

For circular piers the caisson has a diameter varying from 6 to 12 feet; for diameters less than 6 feet it is difficult to excavate the material in the working chamber and, on the other hand, few single-column loads are large enough to require a caisson with a diameter of over 12 feet. The circular caisson is made of staves about 4 by 6 inches in section, usually dressed down to somewhat smaller dimensions, the outer and inner surfaces being cylindrical. The staves are fastened, at every intersection, to inside 3- by 3-inch horizontal angle-iron rings, spaced from 3 to 5 feet apart, bolts of about ¾ inch in diameter, countersunk into the wood, being used for this purpose. The staves are usually splined, but in some cases they are only calked. This type is illustrated in Fig. 119*a*.

In the circular caissons of the Atlantic Mutual Building, which had an average diameter of about 7 feet, the cutting edges were made with a 28- by ⅜-inch steel plate. To give bearing surface to the cutting edge, in order better to control the sinking and to protect the feet of the staves, a 3- by 3- by ⅜-inch angle was riveted to the inside of the plate, parallel to its bottom edge, and ½ inch above it, the horizontal leg forming a shelf to receive the lower ends of the staves. The roof of the

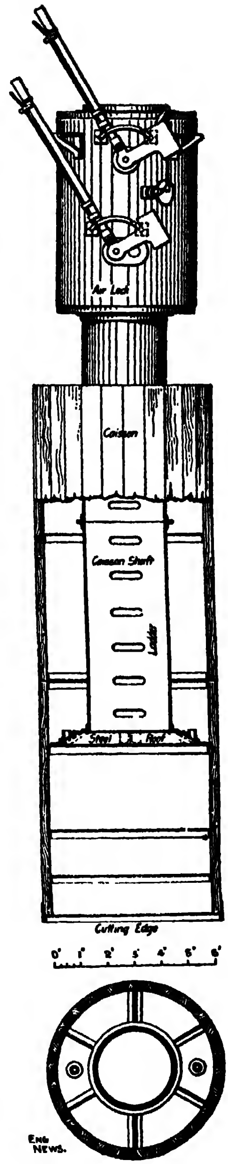


Fig. 119a.—Wooden Stave Caisson with Detachable Roof and Shaft.

working chamber was formed by a removable steel dome ½ inch thick, made in two sections and stiffened with radial steel angles. It was calked with a hemp gasket and bolted to a 3- by 3-inch inside steel angle ring about 6½ feet above the cutting edge.

The crib was of the same form of construction as the caisson, and was built as a continuation of the same to a height of 32 feet above the cutting edge. Where the 32 feet was not sufficient in height, short lengths were added on top. These were made in two semi-cylindrical sections and were butt-jointed to the top of the crib already in place, and were calked and bolted through the horizontal flanges of the angle-iron rings.

For the wall piers of the New York Stock Exchange Building the caissons were made rectangular in form, 8 feet wide, from 24 to 30 feet long and 8 feet high. They were sheathed with 4- by 12-inch vertical wooden staves with square calked edges and without splines. These staves were fastened to successive courses of inside horizontal steel angles, the latter extending wholly around the caisson. The longitudinal walls were braced with horizontal transverse timbers resting on and bolted to the angle frames, as well as with tie rods, parallel and adjacent to the timber braces. The roof was formed with a removable steel-plate dome, reinforced with transverse angles and fastened to frame angles about 6 feet above the cutting edge.

The crib was exactly like the caisson, except that it was without roof or cutting edge. It was built in sections 15 feet high. The angle frames at the top of each section were set 3 inches below the top of the staves with the horizontal flange up. The angle at the bottom of the next upper section had its horizontal flange down and 1 inch below the lower end of the staves. This engaged the lower section and formed a tenon, thus binding the two sections together. A row of eyebolts, 1 foot apart, connected the horizontal flanges of the angle frames.

ART. 120. CAISSONS OF REINFORCED CONCRETE

Pneumatic caissons of reinforced concrete are now being widely used. The chief advantages of this type of caisson are that it gives a monolithic pier, and that the caisson may be

made at the site, thus avoiding the expense of hauling the same. One disadvantage is that the required thickness of walls so reduces the working space that this type cannot be used for very small caissons. Another disadvantage is the time element involved in waiting for the concrete shell to harden.

The foundation caissons of the Municipal Building were sunk in 1910, and were the first in which reinforced cocrete was used throughout. Here both the circular and the rectangular forms were employed; all circular caissons having diameters 9 feet or over and all rectangular ones having a width of 8 feet or over were made of reinforced concrete. Types B and C (Fig. 117*b*) show the outlines of the caissons; it will be noticed that the walls thicken from the cutting edge to the roof by stepping the concrete. As noted in Art. 97, this is a better arrangement than the tapered form because it gives a positive bearing between the chamber shell and the concrete filling, thus making the whole area of the bottom available for carrying the load, without relying on any bond stress. The thickness of the bottom of the wall was about 10 inches and the real cutting edge consisted of a steel channel and a 4- by 4-inch steel angle, the former laid horizontally with flanges up and the latter with its vertical leg down, thus giving the sharp cutting edge and broad bearing surface. The walls were well reinforced with both vertical and horizontal rods.

No cribs were used, simple forms being employed in which to build a concrete shell, which was constructed before sinking was started. The caisson and the shell above the same were built directly on the spot where they were to be sunk. The forms for the interior of both circular and rectangular caissons were made of wood, while for the exterior faces and for the shell above the roof they were made of steel for the circular ones, and of wood for the rectangular ones. In the reinforced-concrete foundations for the Woolworth Building, New York, the inner forms were also of steel for the circular caissons.

Art. 121. Crib and Cofferdam

The frame which is built on top of the caisson and which, together with the roof of the caisson, virtually forms an open box caisson, is generally called a cofferdam when applied to caisson construction for buildings. In the preceding articles, it was designated as a crib, since it corresponds to the crib of the bridge substructure. This frame is usually built in sections, as noted in the preceding pages, and the top section sometimes forms a true cofferdam. As water seldom covers the ground for such caissons, the cofferdam is not often employed; about the only time it being used is when the caisson is sunk before the general excavation for the cellar or subsurface floors is made. In the latter case the cofferdam serves as a form just as the crib proper does, but after the general excavation is completed the cofferdam is removed.

In the early deep foundations, such as those of the Manhattan Life Building, brick masonry was used for the pier material above the caisson, in which case the use of cribs was ordinarily dispensed with, the masonry being built up as the caisson sank. But this arrangement was not entirely satisfactory, for it was found that in omitting the crib the friction on the sides was much increased, which was a disadvantage in itself, and especially dangerous in that it tended to tear apart the brick masonry. Another desirable feature of the crib is that it enables sinking to be carried on without regard to the progress of the masonry construction.

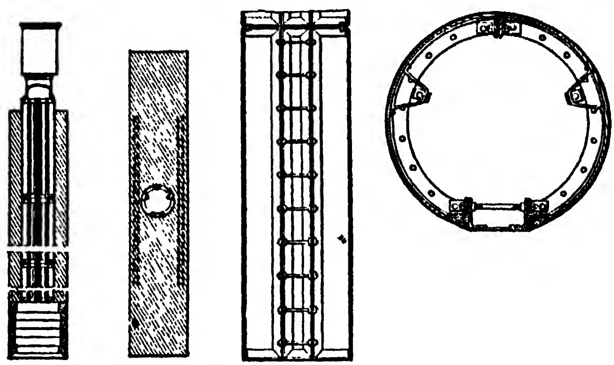
When brick masonry was superseded by concrete, the latter being deposited on the deck of the caisson simultaneously with the sinking of the latter or after it had reached rock, the crib became a necessity. At the present time the tendency is toward the elimination of the crib. As noted in the preceding articles, this is done by building a concrete shell—virtually the pier, except for the hole left for the shafts—before sinking operations are commenced.

If the caisson is not to be sunk over 30 feet, the entire length of shell is cast previous to any sinking, beyond that of pitching

the caisson, that is, sinking the cutting edge a foot or two to give stability; while if the depth is greater than 30 feet, the building and sinking are each done in two operations. This means that the pier is first built up part way, sunk till the top reaches the surface of the ground, then the remainder built and the rest of the sinking done.

ART. 122. SHAFTS AND AIR-LOCKS

Steel shafts are always used in caissons for buildings, and owing to the limited space a single shaft usually serves for both men and materials. For this reason, and for the added one



FIGS. 122*a*, *b*, *c* and *d*.—Collapsible and Removable Shaft.

that it is usually made removable, it differs somewhat from the shafts commonly used in bridge caissons (Art. 106). As noted in Art. 116, in the development of pneumatic-caisson work for buildings the tendency has been toward the elimination of such parts as might weaken the finished pier. The elimination of the permanent steel shaft effected a considerable saving of money, for it made it possible to use the same shafts many times. The first attempts were toward eliminating the steel shafts entirely, not even using the same during sinking operations, the idea being to employ a shaft lining of molded concrete, the latter to be made air-tight by painting. At present this is done to a considerable extent for the lower lengths.

One form of collapsible or removable shaft is shown in Figs. 122*a*–*d*, where *a* shows a sectional elevation of the caisson with

the shaft lining in place, *b* shows a plan of the caisson, while *c* and *d* show details of one section of the lining. [1]"Each section was composed of two approximately semi-circular plates internally flanged for bolting to each other along one vertical edge, and a key interposed between the opposite edges of the plates. Internal flanges at the ends serve for bolting successive sections to each other. Ladder rungs were arranged conveniently between the flanges of the key, and vertical guides were arranged just inside the line of the end flanges to guide the bucket past them."

The shafts should be oiled or otherwise protected from adhering to the concrete. The bottom section of the shaft is usually not made removable, but is thoroughly bonded to the concrete in the crib (see Fig. 117*b*). This is done to prevent the air in the working chamber from leaking between the crib and the air shaft. It also adds resistance against the tendency of the air to blow out the shaft and the air-lock. A somewhat better form of air-shaft than the one just described has an elliptical section in which there is sufficient clearance for a man to pass between the bucket and the ladder. This construction lessens the danger to the men in the working chamber from the lodging of the bucket in the shaft. Such a form of shaft is shown in the lower part of Fig. 122*f*.

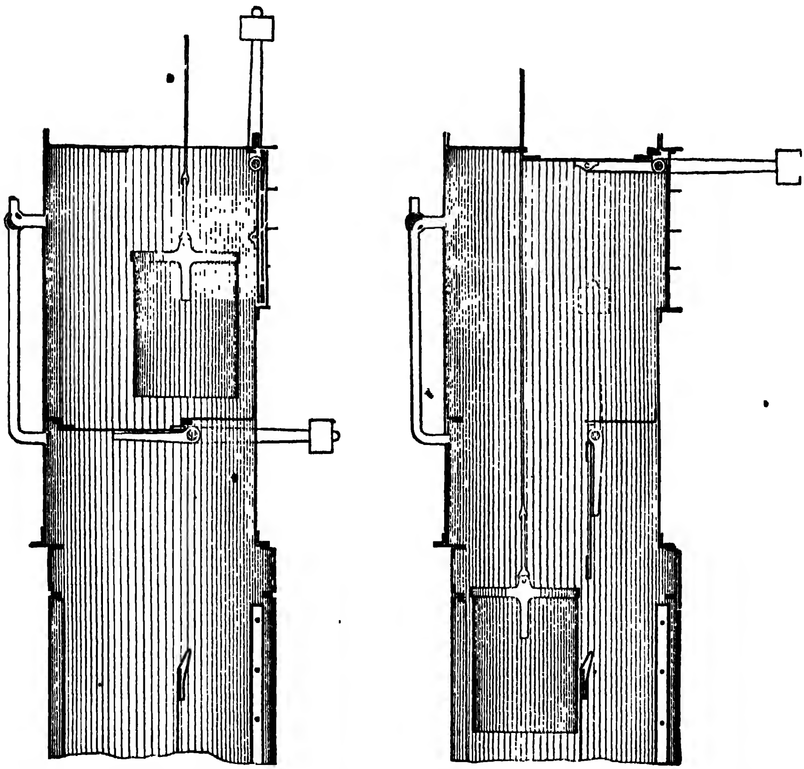
The air-locks are nearly always placed on the top of the shaft, and are made of steel. Two forms called, respectively, the Moran and the Mattson air-lock have been used almost exclusively for work on building caissons. The feature most desired in air-locks for materials is high speed of operation.

Figures 122*e* and *f*[2] illustrate the Moran air-lock for the caissons of the Singer Building, New York. The upper and lower doors are not placed with their vertical axes in the same line. To begin operations the upper door is open and the lower one closed. The bucket is then let down into the air-lock, moved

[1] Recent Developments in Pneumatic Foundations for Buildings, by D. A. USINA, Transactions of the American Society of Civil Engineers, vol. 61, page 219, Dec., 1908.

[2] From Foundations for the New Singer Building, by T. K. THOMSON, Transactions of the American Society of Civil Engineers, vol. 63, page 11, June, 1909.

to one side, the upper door closed, the rope passing through a hole in the door frame, and the valve in the pipe on the left is then placed in the position shown in the illustration; this permits the air from the shaft and working chamber below to enter the air-lock, and, as soon as the pressure in the air-lock nearly equals that below, the lower door opens and the bucket



FIGS. 122e and f.—Oval Shaft Arranged for Men to Pass Bucket. Moran Air-Lock.

is free to be let down. The lower door remains open as long as the bucket is below. On coming out the bucket is raised into the air-lock, the lower door is closed and the valve is turned to connect the air-lock with the outside air, which causes the pressure in the latter to drop to normal; this causes the upper door to open and the bucket is taken out. Both doors are circular,

gasketed steel plates operated by exterior counterweights. The upper door is sometimes provided with a stuffing box to permit the passage of the hoisting rope when the door is closed.

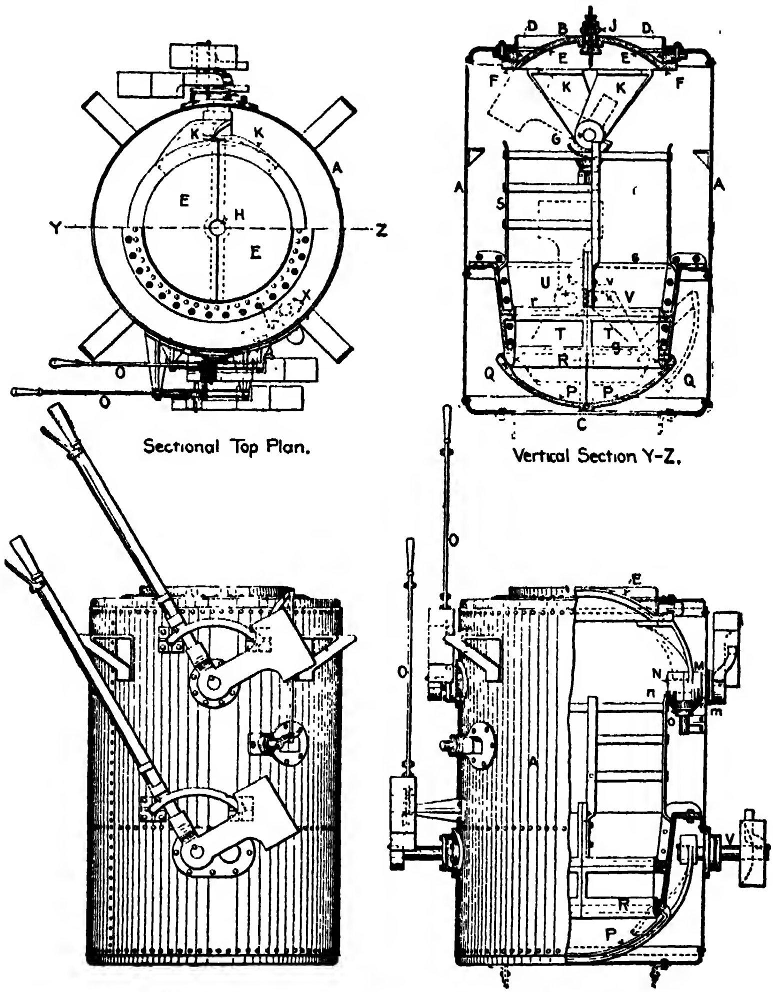


FIG. 122g.—Details of Construction of O'Rourke Air-Lock.

The O'Rourke air-lock is illustrated in Fig. 122g. [1]"Around the top opening is a circular ring D on the inside. This open-

[1] Engineering News, vol. 40, page 364, Dec. 8, 1898.

ing is closed by the pair of oppositely arranged convex swinging gates *E*, the meeting edges of which are packed so as to make an air-tight closure. The opposite edges are provided with flanges *F*, adapted to close against the ring *D*, these flanges having flap gaskets, which protrude into the air-lock so that the air pressure striking them will make an air-tight seal by pressing them against the ring *D* . . .

"The gates *E* are cut away at the center of the meeting edges, as shown at *H*, to receive and fix snugly upon the stuffing box *J*, banded with rubber, and having a hole through the center for the passage of the hoisting rope. The gates are hung by the arms *K* to the common shafts *G*, one (*M*) being fixed to the shaft, and the other (*N*) running loose. This arrangement, by means of the bevel gears and idler *m*, *n* and *o*, allows the two doors to be moved in unison and in opposite directions . . . It will be noticed that the levers have counterweights which balance the doors and thus enable one man to operate the lock.

"The air-lock has its lower end closed by similar oppositely arranged swinging gates *P*, which near their outer edges have seats *Q*, which fit against the ring *R*, with gaskets to secure a tight fit . . . Unlike the upper gates *E*, the lower gates *P* are swung by the arms *T* from separate centers or shafts *U* and *V*. The gate arms are rigidly fixed to the shafts and turn with them. To secure opposite motion to the shafts, one is operated by a spur wheel from the other, as shown at *t* and *v*, the actuating force being obtained through the lever *O*. The admission and discharge of air to and from the locks is controlled by the three-way cock *X*, operated by a lever and bevel gear and connected with suitable piping to the air-shaft, there being no independent connections with the compressor." . . .

ART. 123. SINKING THE CAISSON

Steel caissons are fabricated at the bridge shops, assembled there or at the contractor's yards, brought to the site by trucks, placed in position by derricks and sunk—this is the practice

in New York City. The same general scheme is usually employed with caissons of wood, the main difference being that the material is fabricated in a wood-working mill, because of the limited space usually available at the site, and to the lack of vacant lots in the near vicinity. Some idea of the conditions obtaining at the site in most of the New York City foundation work may be obtained from Fig. 118*c*. As the average caisson with one section of crib seldom weighs over 10 tons, it is not a difficult matter to move them with trucks.

Before sinking the interior caissons the site is usually excavated down to ground-water level; at least, this is true when the cellar floor is to be at or below that elevation. The caisson is then placed and one or more sections of the crib erected on the same, or a section of concrete shell cast if no crib is to be used. The first few feet of sinking is accomplished without the use of air pressure. The material is usually dug by hand and removed with buckets, although the blow-out process is occasionally employed. The disadvantage of the latter process is that the small volume of the working chamber makes it difficult to maintain a constant pressure in the caisson.

One of the gravest problems connected with sinking caissons for buildings is that of safeguarding adjacent buildings form undermining. When a caisson is sunk through quicksand within a few inches of a building, which, perhaps, is founded on a steel grillage, it is evident that great care must be taken not to disturb this quicksand under the grillage. This fact usually precludes the possibility of doing much "blowing," that is, suddenly reducing the air pressure in the working chamber to let the caisson sink a few feet, or of using the water-jet on the outside to reduce friction.

On account of the large friction developed in sinking building caissons—much greater than with bridge caissons, where much of the crib is in water, and therefore not subjected to friction—in addition to excavating the material from the caisson and filling the crib with concrete, special devices much be used to promote sinking. Greasing the sides of the caisson and crib reduces the friction somewhat, and it is usually advisable to do

this. In some cases the caisson may be pulled down by attaching lines to caissons already sunk, or to driven piles, as well as to timbers across the top of the caisson to be pulled down. By far the most effective and customary way is to weight the caisson temporarily with pig iron. At present, either heavy blocks, weighing as much as 4000 pounds each, or ballast boxes filled with pig iron, are employed. A good example of the use of large blocks may be seen in Fig. 118*c*. Some of the ballast boxes hold as much as 12,000 pounds of pig iron. The advantage of the blocks or boxes is that they require no special platform or yokes on the crib, and are very quickly and easily placed and removed by the use of hoisting engines.

Some of the largest caissons sunk to considerable depths have each required as much as 1000 tons of this weighting material, although the average caisson requires about 350 tons. From this it may be seen that for satisfactory cost a means of economically handling this weighting iron had to be developed.

In many of the earlier caissons, such as those of the Atlantic Mutual Building, described in Art. 119, an excessively large amount of temporary weighting was necessary, on account of the concrete not being placed until the caisson had reached its final position. This scheme was adopted in order that the roof of the caisson might be removed after sinking operations were over and the whole pier made a single monolith of concrete. But later caissons have preserved the latter feature without the expense of so much temporary weighting. As explained in Art. 117, this was brought about by using a thin temporary roof, only strong enough to hold a foot or two of concrete on top.

At about the same time that concrete roofs came into use, cribs were largely dispensed with. In their place forms were used, and as these forms were of light construction the concrete was usually deposited in layers a few feet high, and allowed to harden before more was added. As soon as the concrete was sufficiently strong, the forms were moved up and another layer of concrete placed. Where a considerable number of caissons are to be sunk, it has become standard practice to build the

concrete as high as possible before starting to sink. The reasons for this are: first, sinking can be done at a much more rapid rate than can the building of the concrete; second, it saves on the number of times that pig iron must be loaded and unloaded; and, third, it makes less temporary weighting necessary.

Another advantage where the caisson is to serve also as a dam is that it helps eliminate leakage and percolation. Experience has shown that it is almost impossible to make a joint that will not leak.

In the caissons for the City Investing Building the concreting was entirely finished before excavating in the working chamber was commenced, although in some cases caissons were sunk a few feet to give lateral stability to the tall shafts and to relieve the excessive weight on the walls of the working chamber. In the Singer Building, where bed rock was 70 feet below the surface, the concrete was built on the caissons to one-half the estimated total height before sinking was started, after which the caissons were sunk until the top of the concrete was down to the surface of the ground, after which sinking operations were stopped, the remainder of the concrete built and sinking resumed. In some of the piers of the Municipal Building three build-ups were necessary, the maximum height of any one build being 60 feet.

With these high piers great care is necessary in guiding them while sinking. For the caissons of the United States Express Building, heavy horizontal frames, braced with inclined struts, inclosed them. These frames took bearing on greased vertical guide strips attached to the faces of the concrete after the forms were removed.

Art. 124. Rate of Sinking

Although showing large variations, the average rate of sinking caissons in New York City is high, largely because rush jobs are customary there, and also because of the high value of real estate, owners are willing to pay well for keeping the time required for placing the foundations down to a minimum. For this reason many of the records in sinking were not made under

natural conditions, the cost being considerably higher than if more time had been taken.

The caissons of the Manhattan Life Building, which were both circular and rectangular in plan, the former shape averaging about 12 feet in diameter and the rectangular shape about 320 square feet in ground plan, were sunk a distance of 34 feet, mostly through fine sand. This sinking was done, the cribs filled with masonry and the working chamber and shafts filled with concrete, on an average of one caisson in eight days. This corresponds to a sinking rate of 4½ feet per day.

The caissons for the Atlantic Mutual Building (Art. 119) did not have their cribs filled with concrete until sinking was completed. The material penetrated was largely quicksand. One caisson was sunk 24 feet in seven hours. Forty-two caissons were sunk and concreted in 36 days.

In the caissons of the Trinity Building, the average rate of sinking through soft material such as quicksand was about 1 foot an hour, while through hard-pan it was only about one-third as much. These caissons were very similar to those of the United States Express Building (Art. 117), and were built up previously to sinking. The rate as here given refers only to the actual sinking and not to the time spent in building up the caisson, filling the working chamber, etc.

What is probably the best record ever made in caisson sinking was the placing of 87 caissons, all over 75 feet in depth, in 60 days. These were placed for the foundations of the Trinity Annex and United States Realty Buildings by the Foundation Co. of New York City.

Art. 125. Filling the Air Chamber

Where the caisson is to rest on rock, the surface should be thoroughly cleaned of loose and friable material before placing the concrete filling. If hardpan, without any pockets of loose material in it, overlies the bedrock, it is rarely advisable to carry the cutting edge of the caisson more than a few feet into the hardpan. The best method is to stop sinking the caisson

at hardpan level and to carry the excavation below the cutting edge through the hard material down to rock. In this case, when the concrete is placed, it will bond to the hardpan and so reduce the load on the base, whereas if the caisson is sunk through the hardpan to solid rock this bonding effect is lost. Another advantage of stopping the caisson at hardpan lies in the ease with which the bottom section may be belled out to distribute the load over an area larger than the horizontal section of the caisson.

For caissons in which the roofs are to be removed on the completion of the sinking, the working chamber is filled with concrete, which is allowed to harden for about two days, after which the roof and shafts are removed, and the remaining space filled with concrete.

Art. 126. Water-Tight Dam of Wall Piers

In New York, the high value of real estate is resulting in the building of deeper cellars, as many as three floors below street level now being used. Three floors below ground level for the building of the First National Bank of Jersey City resulted in an increased floor area of 25,000 square feet, the foundation work costing less than if individual caissons had been built under the 80 odd columns, which would have been necessary with the lowest floor at street level. These deep cellars demand heavy dam construction around the sides from ground-water level to cellar level. As explained in Art. 116, this dam constuction is obtained by making the wall caissons rectangular in plan and sinking them with a small clearance between the ends of adjacent piers, and afterward filling the space between the piers with concrete or clay to form a continuous and water-tight dam. Some clearance must be left in order to allow for slight deviations in sinking, the usual amount allowed being from 4 to 18 inches. The space between the piers may be made water-tight down to bedrock, in which case the use of the pneumatic-caisson process may sometimes be avoided for the interior column piers, or the space may be made water-tight to a level a little below the level of the cellar floor.

The first building using this form of dam construction was the Commercial Cable Building. The clearance between the caissons varied from 4 to 10 inches. As soon as the caissons were sunk, 3-inch pipes were jetted down in the space between the end walls, and clay pellets were forced through these pipes into the sand by means of a plunger operated by a pile-driver. As clay filled the space, the pipes were gradually raised until the surface was reached, thus forming a water-tight dam of clay. As soon as this was completed, a section of the metal shell in the middle of the ends was removed, the material excavated and the open space filled with concrete, semi-circular wells having been left in the middle of the ends of the caisson.

The caissons for the wall piers of the New York Stock Exchange Building were sunk with a clearance of less than 2 inches, the average being 1 inch. That this is too small a clearance was demonstrated in this work. Water-tightness was obtained in the following manner: As the crib and working chamber were filled with concrete, semi-circular wells were left in the ends. On the completion of sinking the adjoining wooden walls were drawn together and bolted as shown in Fig. 126*c*.[1] The central part of the walls was then removed, thus combining the two wells into one, which was filled with concrete to bond the two piers together.

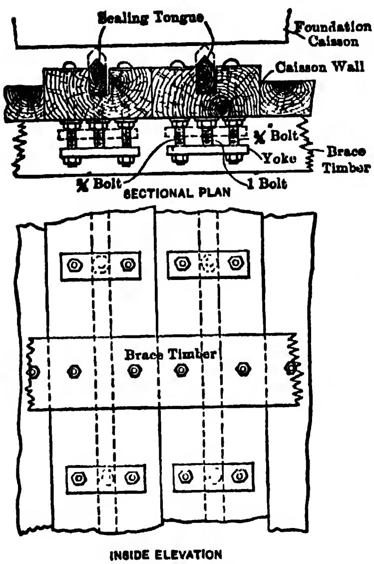
The spaces between the caissons of the Bank of the State of New York Building were sealed by using two 2-inch vertical strips of timber on alternate caissons. These strips were recessed into the wall as shown in Fig. 126*a*.[1] On completion of sinking, the strips were forced out against the adjacent caisson by the simple arrangement shown in the illustration.

The method used for connecting the caissons for No. 42 Broadway is illustrated in Fig. 126*b*.[1]. On completing the sinking, the sand between the guide timbers was removed by jetting the same and the space was then filled with grout.

The method used in the piers of the Trust Company of America Building, where a 12-inch clearance was used, is illustrated in

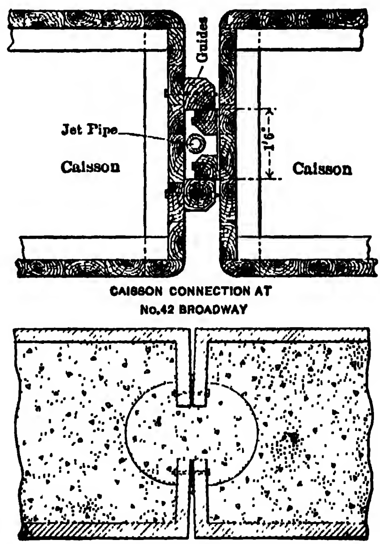
[1] From Recent Developments in Pneumatic Foundations for Buildings, by D. A. USINA, Transactions of the American Society of Civil Engineers, vol. 61, Dec., 1908.

Fig. 126*d*.[1] As shown in section *XX*, semi-octagonal spaces in the center of the ends of the piers were left as wells when the concrete shells above the caissons were built, this building being done previously to the sinking. After sinking the caissons, the earth in the 12-inch space between the cores was excavated to a depth of 1 foot and the upper boards *AA* were removed, cut and placed in the position *A′*. This alternate excavating and



CAISSON CONNECTIONS,
BANK OF STATE OF NEW YORK.

FIG. 126*a*.



CAISSON CONNECTION,
STOCK EXCHANGE.

FIGS. 126*b* and *c*.

sheeting was carried down a few feet, after which the core planks were removed and a short section of a steel air-shaft cylinder set into it and concreted, after which the air-lock was placed on top. The slots *S* were filled with the shaft concrete and acted as keys to prevent the blowing out of the shafts. Air pressure was then put on and the remainder of the material excavated and

[1] From Recent Developments in Pneumatic Foundations for Buildings, by D. A. USINA, Transactions of the American Society of Civil Engineers, vol. 61, Dec., 1908.

boards placed down to the top of the caisson, after which the whole chamber was filled with concrete.

A very neat arrangement was used in the caissons for the U. S. Express Building, where there were clearances of from 6 to

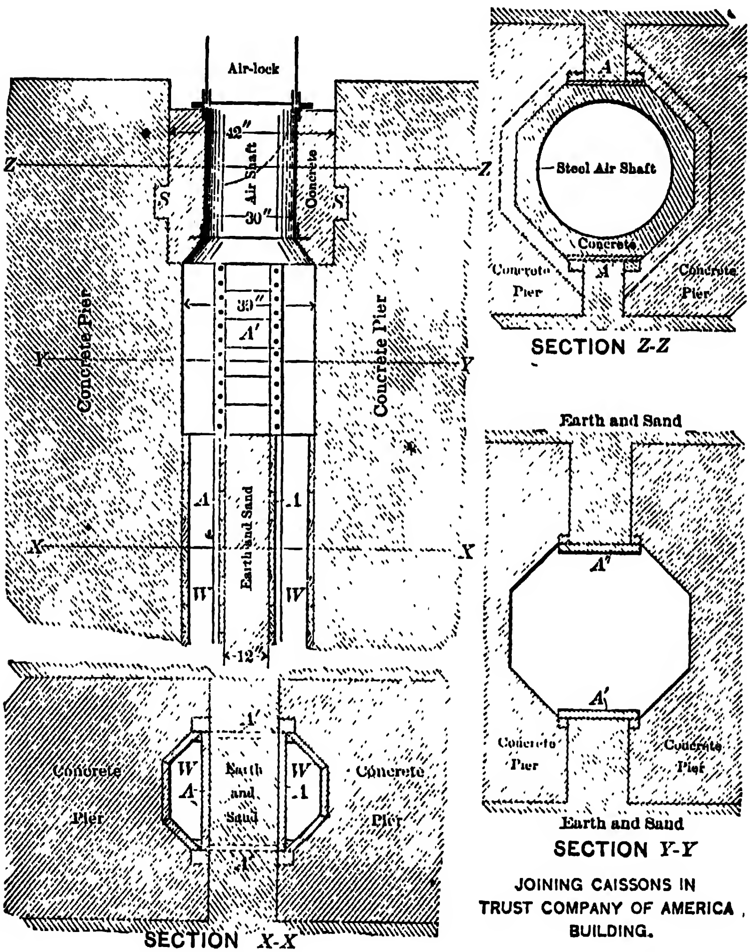


FIG. 126d.—Method of Sinking Joint Well between Caissons.

12 inches. [1]"Vertical grooves about 2 feet wide and 8 inches deep were made in the ends of the wall piers and formed, with

[1] Engineering Record, vol. 53, page 316, March 3, 1906.

the clearances already noted for the caissons, wells from 22 to 28 inches wide above the tops of the working chambers. Compound sheet piles were made with 3-inch planks wide enough to overlap the corners of adjacent piers at each joint and were driven close to the inner and outer faces of the piers so as to cover the joints between them . . .

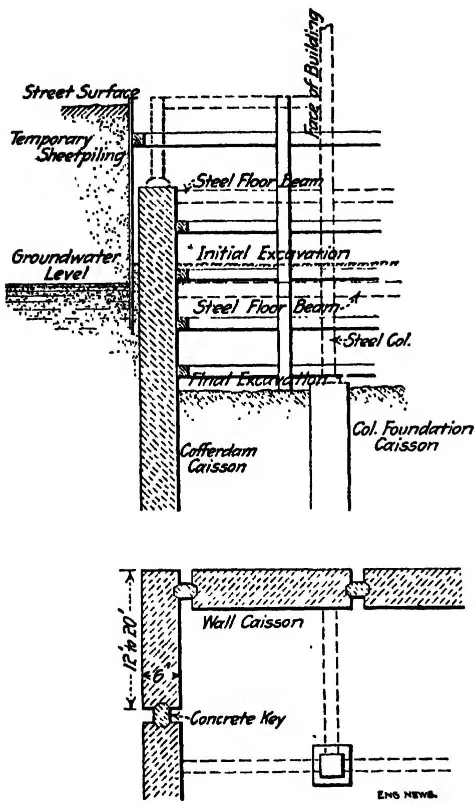


FIG. 126e.—Connection and Bracing of Wall Caissons.

"After the sheet piles were driven, 4-inch pipes were jetted down in the corners between their edges and the outer faces of the piers, and as they were withdrawn grout was forced through them, which effectually sealed the spaces between the piles and the piers. Men were then able to enter the well between the ends of the piers and excavate the quicksand and hardpan down to the tops of the caisson, calking as they went any slight leaks between the sheet piles and the piers. Jet pipes from 2 to 6 inches in diameter were sunk in the narrow space between the caissons and removed or loosened the material down to the cutting edges. Grout was then introduced through them and with the sand and broken stone already there formed concrete thoroughly sealing the space between the working chambers. Afterward the well above the working chamber was rammed

full of ordinary concrete, thus making a solid key which united the wall piers and prevented leakage."

Figure 126e shows a line of wall column piers and the form of bracing usually employed. The whole area of the building is first excavated to ground-water level and sheeted, after which the wall piers are sunk and keyed. The interior is then excavated to cellar-floor level, the wall piers being temporarily braced as the excavation proceeds. The final bracing of these piers is done by means of the floor beams of the building.

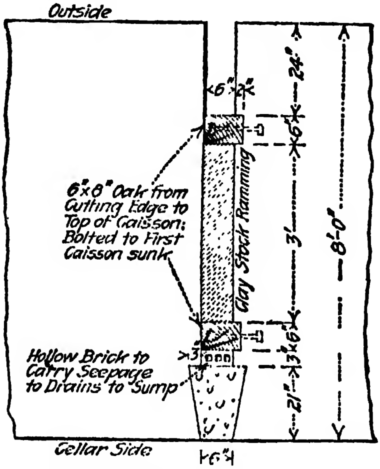


FIG. 126f.—Water-tight Joint between Caissons.

Figure 126f illustrates one of the latest and most satisfactory types of joints. Two 6- by 8-inch timbers are attached to the caisson to be sunk first and extend the entire height of the caisson. After sinking the caissons, a 4-inch steel pipe is jetted down in the space between these timbers and, after the sand has been washed out of the pipe, clay cylinders are rammed down and forced out of the pipe by means of a ramrod, gentle blows from a pile-driver assisting the process. As the ramming progresses, the pipe is raised at intervals until the clay seal is completed to the top. Care must be exercised not to overdo the ramming, else the caissons may be forced apart.

This type of joint has successfully withstood a head of water of 35 feet, while cellar excavation was being carried on as well as the excavation of the sand in the joint as far back as the first timber separator. The latter space was filled with concrete after a layer of hollow brick had been placed to take care of any seepage through the clay fill. According to T. K. THOMSON, this type of joint costs only about one-seventh that of the compressed-air joint, illustrated in Fig. 126d.

CHAPTER XI

PIER FOUNDATIONS IN OPEN WELLS

Art. 127. Open Wells with Sheet-Piling

A method much used for building foundations and occasionally for bridge substructures is that employing the open well. This method gives a type of foundation similar to the caisson, but it is a simpler and more economical process under many conditions. Wells are sunk either by driving sheet-piling and then excavating, or by excavating first and sheeting afterward. The first method is used for quicksand or other material that will not stand up, while the second method is employed in clay, and is known as the Chicago method, because of its extensive use in that city. In either case, as soon as the well is excavated to rock or hardpan, it is filled with concrete to form the pier. The application of the open-well method is limited to those cases in which a moderate amount of disturbance to the surrounding material will not damage adjacent foundations. The sheet-piling method has been used to depths of at least 60 feet and the Chicago method has been used for depths up to 120 feet or more.

The open-well process with sheet-piling, which is used with marked success where rock may be found at moderate depths—from 40 to 60 feet—and where adjacent buildings are not in danger of being undermined, is virtually the cofferdam process applied to building foundations. This process differs from that used for bridge piers in that the sheet-piling usually acts as a form for the lower part of the pier concrete and oftentimes is left in to become more or less a permanent part of the pier.

The wells are either circular, square or rectangular in plan; common sizes are about 6 feet in diameter or on a side, while for rectangular wells the greater dimension is rarely over 16 feet.

Wood, steel, or a combination of both, may be used for the piling. If the well is not over 20 to 25 feet deep the sheet-piling is usually driven in a single section, but for greater depths two or more sections are used, the upper sections being large enough to permit offsetting and placing the lower sections inside.

The upper section of piling is driven first; this may be done by hand or machine. If the driving is not difficult, it is done before excavating is commenced, since there is less likelihood for the surrounding material to be disturbed through flowing into the well from underneath the piling. Great care should be taken to start the sheet-piling in its correct position, as this will save much trouble later. On excavating the wells, which is commonly

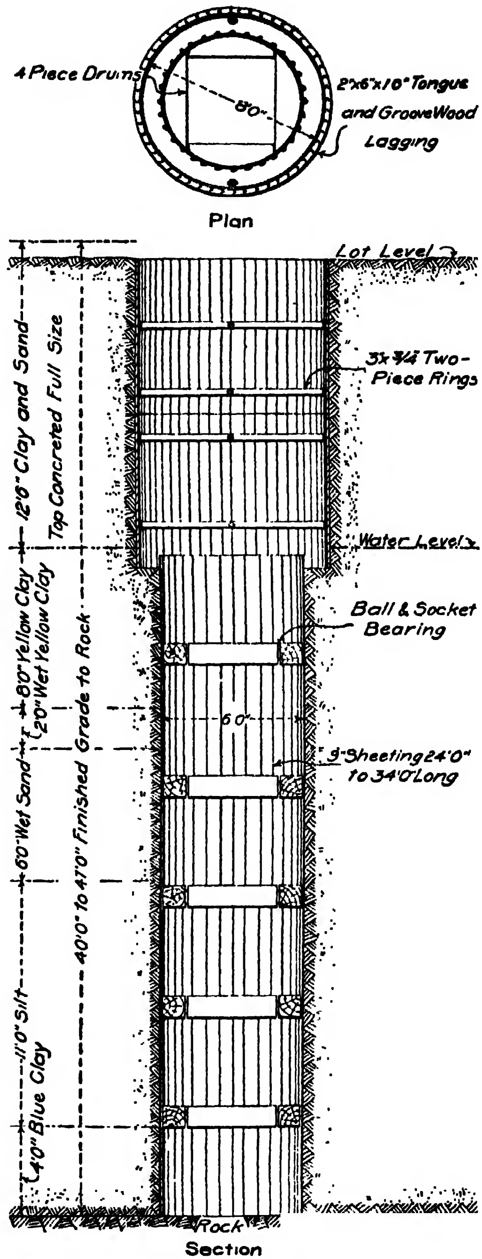


Fig. 127a.—Open Well for Railway Exchange Building, St. Louis.

done by men with picks and shovels, throwing the spoil into buckets lowered into the wells, bracing should immediately be placed.

As soon as the first section is driven and the material excavated, the second section is started. On completion of the work to hardpan or rock, the bottom is carefully cleaned and leveled and the lower section filled with concrete, a 1-2-4 or 1-3-5 mixture being used, and the sheet-piling serving as a form. The latter may be withdrawn after the concrete has set or it may be left permanently in place. If the sheet-piling is to be withdrawn the concrete should be protected in some manner from bonding to it. Above the lower section special forms are usually made for the pier and the whole, including the sheet-piling, withdrawn after the concrete has set.

Figure 127*a* illustrates the cylindrical well sunk to rock for the 22-story Railway Exchange Building, St. Louis, Mo. The illustration indicates the character of the material sunk through as well as the distance sunk. For the upper section, 8 feet in diameter, 2- by 6-inch tongue-and-grooved wooden sheet-piling was used. It was driven by hand and braced by 3- by ¾-inch two-piece rings. The lower section had a smaller diameter and was composed of 9-inch Lackawanna steel sheet-piling, braced with four-piece wooden drums made of 12- by 14-inch material with cast-iron ball-and-socket joints at the ends. After the piling was driven, the material, loosened and kept in suspension by a 1½-inch jet under 100 pounds pressure, was removed with pumps. On completion of the excavation, a 3-foot layer of 1-1½-2 concrete was deposited to seal the bottom, no pumping being done in the meantime. After allowing the concrete to set for five or six hours, the water was pumped out and the lower cylinder filled with concrete, the braces being removed at the same time. The sheet-piling was left in place.

The square piers of the Bamberger Building, Newark, N. J., were built in two sections, with timber sheet-piling above and steel sheet-piling below. They present a good example of very careful guiding of the piling. Each cofferdam was 12 feet square on top and the upper section was lined with 3-inch

tongue-and-grooved planks 20 feet long. The piling was assembled on horizontal skids to make panels 12 feet wide with transverse cleats on top and bottom.

A pit was first excavated and in it was placed the bracing frame shown in the right-hand drawing of Fig. 127*b*. The three sets of horizontal 10- by 10-inch frames were braced together with diagonal planks and the two upper frames rested on 10- by 10-inch posts 4 feet long. The ends of the rangers were halved and were connected by short 2- by 10-inch planks.

"The side panels of vertical sheeting planks are lifted by a derrick, the tackle being attached to a bridle connected to the ends of a pair of horizontal planks tightly clamped to the upper

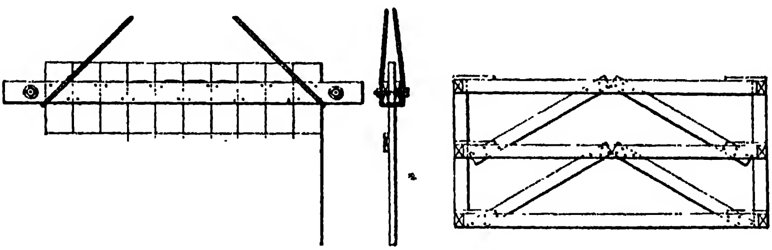


FIG. 127*b*.—A Side Panel and Frame of Cofferdam.

end of the assembled panel (see left-hand illustration, Fig. 127*b*). After the four panels are set in place against the faces of the rangers forming the interior framework they are secured by light yokes of horizontal timbers and tension rods screwed up tight, after which the temporary cleats are removed and the sheeting is driven by light steam-hammers as the excavation progresses inside, the rangers being forced down as necessary."

Figure 127*c* shows the sinking of cylindrical wells for the foundations of the Kinney Building, Newark, N. J. After being assembled by stiff-leg derricks, the steel-piling units were clamped together by outside wire cables holding them against inside ranger frames spaced from 2¼ to 5 feet apart and made from two thicknesses of 3- by 5-inch scarfed planks with five to

[1] Engineering Record, vol. 64, page 457, Oct. 14, 1911.

seven pieces in each course. The piling was driven to bottom before any excavating was done and was removed after the well was filled with concrete.

In placing cylinders varying from 33 to 81 inches in diameter and 35 feet below ground surface for a building in Tonawanda, N. Y., a timber pile was first driven on the center line of each pier. The material around the pile was then excavated to water level and a templet set in position. A timber tower about 35 feet high was then built and an upper templet, 22 feet above the lower one, placed and held in position by the tower. A 14- by 14-inch timber mast was then mounted and held in position on top of the wooden pile by a 2-inch steel pin and guyed at the top.

All the steel sheet-piling was placed in position and interlocked by means of a cableway with two timber portable towers 70 feet in height, the position of the cableway permitting all the piles in one cylinder and all the cylinders in one row to be assembled without moving the tower. A steam pile-driving hammer was then supported from the mast by a steel A-frame, the hammer being free to slide up and down and the mast free to revolve. Driving was then started, two piles being driven 3 or 4 feet at a time and then the next two, and so on until all the piles reached bed rock.

The material in the cylinders consisted of mud, silt, sand and, near the bottom, clay containing quantities of glacial drift. This material was removed by a sand-jet pump consisting of a 40-foot length of 6-inch pipe with a elbow at the top to which was connected a horizontal 10-foot length of pipe. Running down the 6-inch pipe on the outside were two jet pipes with reducers at the end. One jet pipe was turned upward at the lower end and toward the center of the 6-inch pipe, while the other was straight and extended about 1 foot below the bottom of the 6-inch pipe. A third jet was placed at the elbow of the large pipe. The latter pipe was then placed on the material in the cylinder and the jets turned on after closing a gate valve at the elbow of the 6-inch pipe. This forced all the water downward, resulting in the rapid sinking of the pipe. After

FIG. 127c.—Sinking Open Wells for Foundation Piers of Kinney Building, Newark N. J.

(Facing p. 400.)

FIG. 128a —Open Well with Sectional Lining of Corrugated Iron. Baltimore & Ohio Railroad Bridge over Calumet River, Ninety-Second Street, South Chicago, Ill.

it reached bottom the valve was opened and in less than two hours all the sand and loose material had been removed.

On completion of excavation a cage of reinforcement was placed in the cylinders, after which the cylinders were filled with concrete by the tremie method.

Art. 128. Open Wells with Sheeting: The Chicago Method

The soil conditions in the downtown or business district of Chicago, where most of the heavy buildings are located, are peculiar and have led to the use of a special type of foundation for many of the heavy structures. For a distance of about 14 feet below the street curb the soil consists of loam and made ground; below this there is a layer of clay having a thickness of from 70 to 80 feet, which overlies hardpan or coarse gravel and solid rock. The upper 6 to 12 feet of this clay is hard and stiff and forms the bed on which rest many steel grillage foundations (Arts. 155, 156, 160) which, dating from 1878, were so extensively used in that city. Below this, the clay becomes softer and remains so down to the hardpan, which has a thickness of from 10 to 20 feet. In general, this softer clay differs from that above only in the larger amount of water contained in it. In places pockets of quicksand are present in the soft clay.

The clay is sufficiently stiff to permit sinking wells by excavating the clay in sections about 4 feet deep, each section being sheeted with 2- by 6-inch or 3- by 6-inch planks in 4-foot lengths as soon as the section is excavated. In some cases the sheeting has been made of sheet metal. The wells vary from about 3 to 12 feet in diameter.. Thus, this method differs from the sheet-piling method essentially in that the excavation is made before the lining is placed, while in the sheet-piling method the lining is always placed in advance of the excavation.

Digging the Well.—The wells are excavated by hand to the required diameter, from one to four men working in a single

well. As soon as a section is excavated, tongue-and-grooved lagging, 2 or 3 inches thick and not over 6 inches wide and beveled to form a true circle, is placed. Two or perhaps three iron hoops, ¾ by 3 inches in section, or angle-iron hoops, are used to brace the sheeting of each section. These hoops are made in semi-circular form with their ends bent inward to form flanges which are bolted together as shown in Fig. 127*a*. As soon as the bracing for one section is placed the next section is excavated and the lagging for that section, abutting against the lagging for the section above, is placed, and this cycle is repeated until the hard material is reached.

Care must be exercised to have the lagging fit tightly against the clay in order to prevent any flow of the same. As this is somewhat difficult to accomplish with the above-described type of hoops, another form has been invented by G. W. JACKSON. Each brace consists of four sections of steel T-bars bent to form a circular sectional rib bearing against the lagging, and of a hollow central hub to which are attached jackscrews radiating from the hub like the spokes of a wheel. The heads of these jackscrews are fitted to shoes on the horizontal web of the circular rib or rim. As many jacks as necessary may be used, but not less than four, one for each section of the rib. The jacks may be set up to compress the surrounding material as much as desired.

The spoil is removed from the wells by buckets operated by a windlass or other arrangement. For small jobs the windlass may be worked by hand, but where a large number of piers are being sunk power is used. The Thomas Elevator Co. of Chicago build a multiple spool hoist which will operate a number of wells by one motor, each one independently of all others.[1]

The lagging and bracing are sometimes removed as the concrete is placed, but if the surrounding material is at all soft they are usually left in. A 1-3-5 mixture of concrete is commonly used for the filling.

Where the pier rests on hardpan, the lower part is ordinarily belled out to about twice the diameter of the pier, the belling

[1] See Engineering News, vol. 65, page 133, Feb. 2, 1911.

being done at an angle of approximately 45 degrees. The unit bearing pressure allowed is about 7 tons per square foot for hardpan and about 30 tons for rock.

APPLICATIONS.—The first building in Chicago and the first in the United States, with the exception of the City Hall of Kansas City, to have this type of foundation was the Chicago Stock Exchange, built in 1892. This structure was founded on piles and on piers sunk by the Chicago method, the latter being used where it was feared that the jarring of pile driving would disturb the foundations of adjacent buildings.

For the wells of the City Hall foundations in Kansas City a metal shell lining was used instead of wooden lagging and the piers were constructed of brickwork instead of concrete.

The foundations for the new City Hall of Chicago were composed of circular concrete piers from 4 to 10 feet in diameter and seated on bed rock 96 to 120 feet below street grade. Three-inch tongue-and-grooved lagging in 4-foot lengths was used. The clay spoil was dug by hand, one to four men working in a well at one time. The buckets held 3 or 4 cubic feet and were raised and lowered by means of timber tripods set up over the wells. A drive wheel was placed on one side of each tripod and was connected to a shaft that carried a winding spool. A single endless cable on a hoisting engine connected with a number of the driving wheels—the tripods being set up in straight rows—and thus readily served seven or eight wells.

In applying the Chicago method, modifications may be made to suit local conditions; for instance, the sheet-piling and the sheet method may be combined in the same well. This was done in the foundation work for the Hotel Brevoort, Chicago, where the presence of a high building nearby made necessary the use of steel sheet-piling for a depth of 30 feet, while below this the ordinary lagging was used.

Figure 128*a* shows the details of the 65-foot wells used for the foundations of a double-track bascule bridge of the Baltimore and Ohio Railroad in South Chicago. The surface of the ground was at about water-level and for the upper 18 feet the

material was quicksand, there being below this an impervious stratum of soft blue clay which extends to rock.

A cofferdam made of 3- by 12- inch tongue-and-grooved sheet-piling in 21-foot lengths was driven through this and to the stiff clay and braced with three tiers of 12- by 12-inch rangers, with 45-degree knee braces at the corners. The bracing was placed as the well was excavated. At the bottom, a 12-foot-diameter well was sunk and lined with lagging composed of courses of No. 20 corrugated iron in 2-foot widths and of lengths equal to the circumference of the well. Vertical 2- by 2-inch flange angles about 23 inches long were riveted to the rims of both ends of each section and through open holes in the outstanding legs were bolted together when placed in position to make complete rings.

After placing the first section, 12 inches of concrete was deposited between the same and the lower part of the cofferdam to seal the space between the cofferdam and well. As the material was excavated, additional rings of lagging were placed, each one overlapping the one above it by a single corrugation. No bolting of horizontal joints was done and no bracing was used.

A 10- to 12-foot layer of quicksand with its surface 100 feet below the street curb was struck in sinking the wells of the Chicago Edison Co's. building. Below this there was a layer of boulders overlying the bed-rock and these boulders varied from cobble-stone size to 5 feet in diameter. On reaching quicksand the usual method was abandoned and steel cylinders in three sections, with vertical joints flanged with angle-iron connections, were sunk. As the quicksand was removed from the interior these cylinders sank by their own weight until the boulders were reached. These had to be drilled and split open to permit the caissons, aided by jacks, to pass through.

In the wells for the foundations of the Northwestern Railroad Terminal[1] the pneumatic-caisson process was used when a heavy water-bearing stratum just above rock was struck.

[1] See Engineering News, vol. 62, page 554, Nov. 18, 1909.

ART. 129. THE GROUTING PROCESS

The general idea of the grouting process is to inject fluid cement between and among materials already in place and thus cement the mass into a solid concrete. The process may be used for forming new foundations or for repairing old ones. For foundations on land two general methods may be used: The whole foundation bed, down to rock or other firm material, may be turned into concrete *in situ* and the piers built directly upon it; or a ring of concrete may be formed around the site, forming a sort of cofferdam, after which the interior may be excavated down to solid material and the substructure built within it. The latter method will give a more reliable foundation but a more expensive one. In using the former method it is a difficult matter to prevent pockets of uncemented material from being present. It seems that this process may be used satisfactorily for any material varying from the size of broken stone down to fine sand. Clayey material cannot be grouted.

Two methods have been developed for the application of the grouting process: the first uses the cement in the form of a fluid, and the second in its dry state. The first may be subdivided into two methods, one being used where fine material is encountered and the other for coarse material.

Where cement grout is used in fine material, two pipes, a short distance apart, are first driven. Water is then pumped down one pipe and in taking a course of least resistance will come up the other pipe, thus cutting out a channel between the two pipes. By using a number of pipes as many channels as desired may be made. As soon as a well-developed channel is formed, cement grout is pumped through the pipe instead of the water. When the grout appears in the outlet pipe the latter is closed by a valve and the pumping continued, thus forcing the grout to permeate the sand around the channel. In this way a stratum of solid mortar or concrete is formed; by employing the same scheme at various depths the whole mass becomes solidified.

Where medium-sized material is encountered it is often only necessary to drive a row of pipes and pour the grout into them, the head being sufficient to force the grout throughout the material.

In coarse material the difficulty lies in keeping the grout within bounds and preventing it from spreading out in thin layers and running into adjacent territory, or below the level at which it is desired to form the concrete. This difficulty may be overcome by using the principle of successive accretions. In using this method only a small amount of grout is poured into any one pipe at a time. After this has had time to set, more grout is poured in and the operation repeated until a solid floor of concrete is made. Walls may be made in the same manner, after which the interior may be filled with grout or excavated.

The method which employs dry cement is as follows: The cement is blown through a 1½-inch pipe, drawn to a point at the lower end, in which there are three or more holes of about ⅜ inch diameter. The pipe is free to be raised or lowered, and is connected at its upper end with an air-pressure supply pipe. To this air-pressure pipe, suitable connections are made of suitable branches, stop cocks, etc., and by means of an injector cement powder is fed into the air current. The cement powder, by means of an air current, is forced through the small openings in the lower end of the pipe and is driven into the sand. In consequence of the boiling action caused by the air bubbles running through the water in the sand, the cement is thoroughly mixed with the latter, and as soon as injecting is stopped the sand with the particles of cement clinging to it settles into place and forms concrete or mortar. The volume of cement used should be about one-fifth the volume of the sand. In fine material each sinking of the pipe will cover about 1 square foot of ground and the cement must be forced out at different elevations, the pipe being slowly drawn up as the cement is blown into the sand. The dry method is seldom used at present. For further details concerning this method the reader is referred to an article by FR. NEUKIRCH in the Transactions

of the American Society of Civil Engineers, vol. 29, page 639, Sept., 1893, entitled Improved Method of Constructing Foundations under Water by Forcing Cement into Loose Sand or Gravel by Means of Air Pressure.

ART. 130. APPLICATIONS AND TESTS

The exclusion of water from the site is one of the most expensive items connected with foundation work. The economy of the grouting process lies in the fact that the necessity for doing this may be avoided or else it may be done cheaply.

Grouting has been used quite extensively for repairing dams, quay walls, etc., where the water has washed out the filling. In such cases it is customary to sink pipes and pour cement grout into them, the pressure head on the grout being sufficient to force it into place. It has been found that the pressure head of a column of grout is about double that of water. For an example of the use of the grouting process for the foundation of a cylinder caisson, see Art. 105.

The left abutment (on land) of a concrete arch bridge at Ehingen, Germany, was founded on a bed of concrete formed in place by the injection of grout. The material was water-bearing gravel. [1]"Twelve-foot lengths of 1½-inch pipe, with an iron driving point loosely inserted in the lower end, were driven down to rock, and then, by raising a few inches, lifted clear of the driving point. Cement grout was then pumped in until a rapid rise in pressure indicated saturation; the pipe was then drawn up a small distance, grout pumped in again to saturation, and so on." Pipes were driven at intervals of 18 and 20 inches and test excavations made afterward showed a very good quality of concrete.

[1]"To found the two river piers, cofferdams of sheet-piling were driven to rock and made water-tight by injection of cement through pipes driven around them. In the case of one of the piers, pipes were driven inside as well as outside, with the result that nearly the whole mass in the interior of the cofferdam was

[1] Engineering News, vol. 47, page 35, Jan. 9, 1902.

cemented into a block of concrete. On account of some layers of sand, however, about one-half of this mass was broken out again and the pier regularly built up of concrete above the remaining conglomerate. At the other pier, the cementing was carried out only around the outside of the cofferdam, making it perfectly tight. The interior was then excavated and the concrete of the pier built up directly on the rock."

A good example of the successive accretion method of grouting is that of rebuilding one of the piers of a bridge on the New York Northern Railroad, across Croton Lake, N. Y. The pier was about 22 by 32 feet in plan and about 7 feet high. It rested on a crib 35 by 47 feet in plan, made of 4-inch planks laid cob fashion, and divided into nine compartments, filled with stone. The top of the crib was about 5 feet below water. The problem was to make a solid pier out of the one which, as originally built, was not filled with masonry but with rocks, sticks, dirt and all sorts of rubbish, there being merely a shell of masonry around the outside. As stated by the engineer, R. L. HARRIS: "We wanted a tight bottom at any level below the top of the crib and tight sides thence to the water surface. The idea was to use the materials that were in place, and make a caisson therein without disturbance, by cementing, for the floor of the caisson, a portion of the loose mass of irregular stone filling in the crib at any level below the top of the crib; and for walls, to cement from thence to the water surface, or as high as necessary to make a good connection with the shell; this could then be pumped out, the interior carefully excavated to the crib, and the space filled with concrete rammed in layers to the top of the old shell."[1]

Some of the interior masonry was removed from the top and then holes were worked among the stones extending a few feet below the top of the crib. "A long nozzle of 1½-inch iron pipe, connected to the discharge pipe of a No. 2 Douglas hand force pump, was inserted in one of these holes to its bottom, water was rapidly pumped through for a few minutes, then the suction

[1] A Cofferdam without Timber or Iron, by R. L. Harris, Transactions of the American Society of Civil Engineers, vol. 24, page 234, March, 1891.

hose was suddenly transferred to a reservoir of grout, composed of portland cement and fine sharp sand, in equal parts, mixed immediately before use; a small quantity only of the grout was slowly forced through, and the nozzle was then withdrawn but the hole maintained, and the same operation was continued at other holes, seldom returning to any hole the same day; the belief being, that in quiet water the cement would accrete on the surface of irregular stones at and below the level of the injection, and that by consecutive slight accretions at proper intervals of time the voids between them would be filled." The results were successful.

TESTING THE GROUTING PROCESS.—Although many examples exist of the successful application of this method, yet there

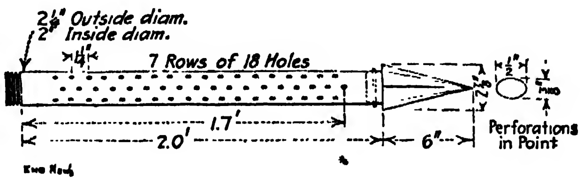


FIG. 130a.—Well Point on End of Grouting Pipe.

is always some uncertainty regarding the degree of success in any particular case. To test its reliability the Louisville and Nashville Railroad had some interesting experiments made, a complete description of which may be found in Engineering News, vol. 69, page 979, May 8, 1913.

The most interesting experiment made was in gravel, where bed rock was 23 feet below the surface and the water-level 8 feet below. An analysis of the gravel gave the following percentage in its mechanical composition: coarse gravel, 12; very fine gravel, 34; sand, 44; and silt, 10. Two-inch pipe in 5-foot sections and with a well point, illustrated in Fig. 130a, were driven to rock on the circumference of a circle 15 feet in diameter and with a spacing of about 3 feet. An average unit pressure of 20 pounds was sufficient to force the grout into the gravel, although at times 60 pounds would not clear the pipe. The pipe was slowly withdrawn as the grout was forced in, the one operation

following the other, enough grout being forced in to fill all voids for a distance of 2 feet out from the center of the pipe. On allowing the material to harden and then excavating the core, it was found that the wall was sufficiently good to allow the water to be pumped down within 2 feet of the rock.

Figure 130*b* shows the appearance of the concrete after removing the core. The grout followed the path of least resistance which was essentially upward. In coming up, much of the silt in the gravel floated on the grout. In addition to the latter

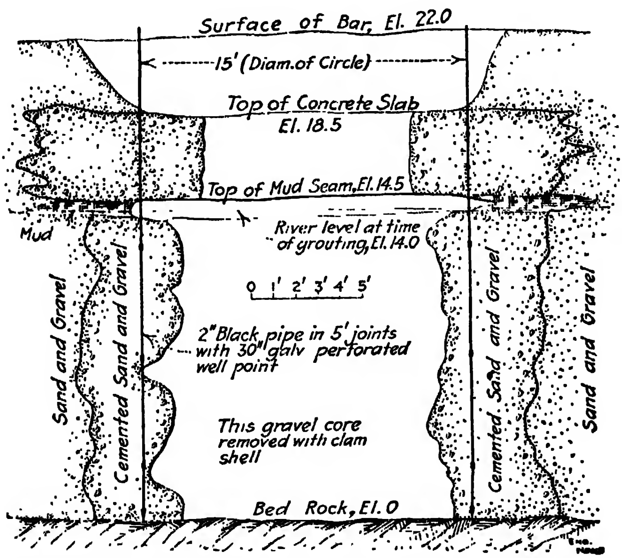


FIG. 130*b*.—Typical Cross-Section of Concrete Cylinder Formed by the Circle of Grouting Pipes.

effect the grouting below tended to disturb the material at the top causing the fine material there to separate from the coarse, thus leaving a very porous layer just above water-level, with a layer of silt at water-level. As a consequence, for 4 feet above the water surface the best concrete formed and this extended across the cylinder and had to be dug out with picks on excavating the core. On the other hand, at the mud seam no concrete formed, even at the pipe. Below the water-level the concrete was irregular and not especially good.

ART. 131. THE FREEZING PROCESS

The idea of freezing the soil, as an aid to excavation, has existed for many years, and although it has attained a considerable degree of success in the sinking of mine shafts, particularly in Germany and other foreign countries, it has seldom been applied to foundations. However, owing to the inherent possibilities of this process for foundations at great depths, the principles are worthy of careful study.

The presence of water causes the principal difficulties in foundation work, especially when water is present in very fine sand, forming what is known as quicksand. If the water can be frozen the work becomes easy. In the method invented in 1884 by F. H. POETSCH, M. D., a Prussian, tubes are driven around the outside of, or into, the soil, all over the site to be excavated, and a freezing mixture is made to circulate through these pipes, which gradually transforms the soil into a non-water-carrying solid mass, after which the excavation can easily be made. If the pipes are driven to a non-water-bearing stratum it is only necessary to freeze a wall around the site, but if an impervious stratum is not reached the whole site, or a ring around the site and a layer of soil near the bottom, must be frozen.

Long water-tight tubes closed at the bottom, from 4 to 6 inches in diameter and spaced about 3 feet apart, are first driven through the mass to be frozen. Inside of these tubes are placed small pipes, from 1 to 1½ inches in diameter, which are open at the bottom or have openings in their sides near the bottom. A considerable number of the small circulating tubes are joined together by a larger pipe, and the larger or freezing tubes are capped and joined together by another pipe. A circuit is then formed and cold brine is drawn from a tank, pumped down the circulating tubes, up through the freezing tubes, and back to the freezing machine. For shaft sinking the pipes are usually placed around the circumference of a ring with perhaps a few inside which are so insulated that they freeze only the bottom of the shaft.

What is said to be the first application of this process to building foundations is that for the substructure of a depart-

ment store in Berlin. Figure 131*a* illustrates the conditions obtaining at the site as well as the general plan of the process. The subsoil was a quicksand with ground-water level about 13 feet below the curb. The foundations of adjoining buildings were 10 feet below the curb, while the excavation for the new

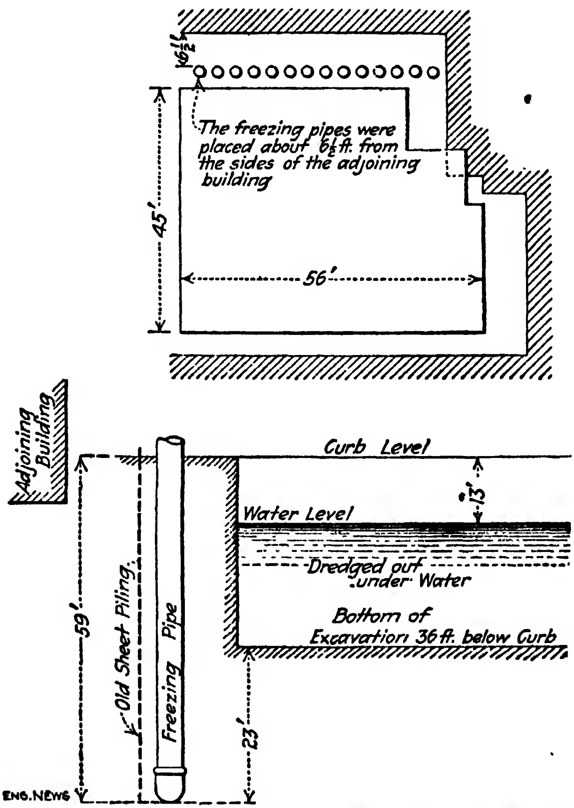


FIG. 131*a*.—Building Foundation Constructed under the Freezing Process.

structure had to be carried to a depth of 36 feet below the curb. Sheet-piling was first used, but as soon as the excavation reached below water-level the sand from under the adjoining buildings on one side of the lot commenced boiling up in the excavation, causing several structures to settle and crack. The freezing process was then adopted. Freezing pipes 5 inches

in diameter and about $\frac{5}{16}$ inch thick were sunk on 3-foot centers, as shown in the illustration, and extended 59 feet below curb level. The circulating pipes were 1 inch in diameter and were connected to a supply header at the top, while the 5-inch pipes were connected to a drain header. The liquor passed through the circulating pipes with a velocity of $11\frac{1}{2}$ feet per minute. About four weeks after the brine was started the ground was frozen a sufficient distance to begin excavating, after the completion of which the foundation was placed. The cost is said to have been lower than if the pneumatic-caisson process had been employed.

The freezing process was successfully applied to a leak in the cofferdam for the west river pier of the Detroit-Superior arch bridge over the Cuyahoga River at Cleveland. The head of 46 feet caused the steel sheet-piling to bulge badly, which resulted in the leak. The cost of stopping the leak by this method was about \$1600. Before using the freezing process about \$10,000 had been expended in trying to stop the flow of water by other methods.

Only under special circumstances, or where no other process can be adopted, or where a refrigerating plant is located nearby, will the freezing process prove commercially practicable.

ART. 132. HYDRAULIC CAISSONS

This type of caisson has been used in a few cases for deep building foundations but it is ill adapted to most soils. Where sand predominates and no boulders are present, it may be used with success. The caisson consists of a riveted steel cylindrical shell, say from 5 to 14 feet in diameter and as high as necessary. The lower edge is shod with a hollow cast-iron cutting edge of a triangular cross-section, which is perforated with many holes forming special nozzles. This cutting edge is composed of a number of sections, each section having an inside chamber independent of all other sections. By means of pipes and flexible tubing these chambers are connected with a force pump. The material is first excavated to ground-water level, after which the caisson is placed in this excavation; the caisson is

then heavily weighted and water is forced into the cutting-edge chambers and thence out through the small nozzles to scour the material from under and around the cutting edge, thus causing the caisson to sink. When the stratum on which the caisson is to rest is reached, the hydraulic pressure is discontinued and the spoil is excavated from the interior in the dry, after which the pier is built by filling with concrete. If the caisson is bedded in clay, the excavating and pier building are easily done in the dry, but if it rests on rock it is often a difficult matter to keep out the water. This feature and the risk of meeting boulders in sinking make this method of founding piers very uncertain. This type of caisson was used in placing the foundations of the Johnson and the Meyer-Jonassen Buildings, both located in New York City. Descriptions are given in the Engineering Record, vol. 32, page 116; and vol. 33, page 315. It appears that the use of this method has been abandoned.

CHAPTER XII

ORDINARY BRIDGE PIERS

Art. 133. General Requirements

In selecting the site of a bridge and arranging the piers, careful attention must be given to such matters as location of crossing, position and spacing of piers and abutments, height of bridge, required waterway, etc. Where the construction is in new country, the location of the bridge can usually be made to suit the engineering requirements. These will be best satisfied where the width of the river is not great; however, it should not be located in the narrowest part, for there the current is apt to be swift and the water deep in heavy rains, thus making the construction of the substructure both difficult and expensive. On the other hand, where the bridge is located in a built-up community, it will have to be placed where it will best serve the needs of the people. If it is a highway structure it will connect main thoroughfares on the two sides of the river, while a railroad structure has to connect the rights-of-way. Building new streets or buying rights-of-way is very expensive in built-up vicinities and will usually be in excess of any possible saving in the cost of the bridge by placing the latter in a more advantageous position from an engineering standpoint.

In determining the number of piers and their spacing, due regard should be given to the financial considerations, the navigation interests, waterway requirements and the Government rules and regulations.

The financial requirements are best served by an arrangement which makes the total cost of the bridge, superstructure plus substructure, a minimum. As the cost of the superstructure, exclusive of floor system, varies approximately as the square of the length of a span, and the cost of a pier with its foundation is approximately a constant for fairly wide ranges of span

length, there is some length of span which, with its corresponding number of piers, will make the total cost of the bridge a minimum. For the deduction of such a formula, see ART. 9, of MERRIMAN AND JACOBY'S ROOFS and Bridges, Part III. This formula shows that for minimum cost the cost of one river pier should equal the cost of the main and lateral trusses of one span. In deriving the formula it was assumed that the lengths of all spans are approximately equal.

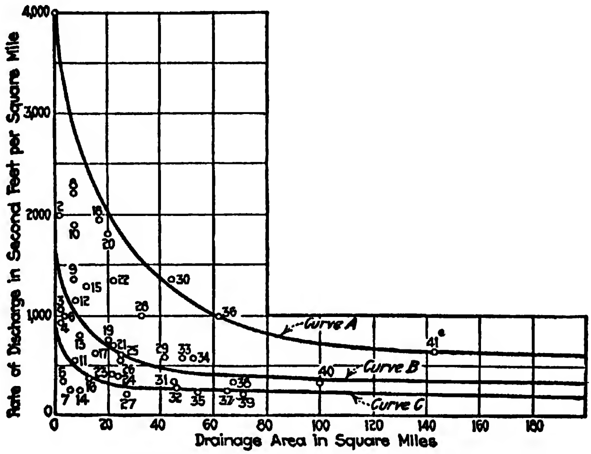


FIG. 133*a*.—Rates of Discharge from Small Drainage Areas.

Figure 133*a* shows waterway curves recommended by IVAN E. HOUCK,[1] the ordinates representing the rate of discharge in cubic feet per second per square mile and the abscissae the drainage area in square miles. Each small circle represents an actual run-off record.

Except where the channel cross-section is unusually large, or where damage from a failure of the structure would be great, it would in general be impracticable to design waterways as large as required by curve *A*. However, where possible, water-

[1] Engineering News-Record, vol. 88, page 1071, June 29, 1922.

ways should be designed to meet the requirements of curve *B* and in no case should values be provided for less than shown in curve *C*. A mean velocity of 10 feet per second may safely be allowed under the bridge.

Navigation interests require that the piers shall be placed so as to cause as little danger and obstruction as possible to river traffic. Thus they should be kept out of the channel and should be spaced at considerable distances apart.

The pier should rest on a stable, unyielding foundation, the base of which is well below the frost line and below the elevation of any possible scouring action. Where rock or other satisfactory bearing material lies at a depth not greater than from 20 to 30 feet below water-level, the pier footing will usually be placed directly on the rock surface, a cofferdam being used if necessary. The material overlying the rock is first removed, after which the latter should be leveled or stepped off and cleared of all loose material before placing the footing for the pier.

For depths varying from 20 to 40 feet or more a pile foundation will usually prove the cheapest. The correct principles of design for this type of foundation are discussed in preceding chapters. For depths greater than about 40 feet, some type of caisson foundation is generally used.

Shallow foundations, corresponding to the spread footings so much used for buildings, are seldom used for bridges. Up to about 30 years ago, a spread footing consisting of a timber grillage was a common type of foundation for bridges. The grillage consisted of a more or less open mass of timbers laid directly on the gravel bottom after dredging out a few feet, and extending to nearly low-water level. The grillage was built, with courses alternating in direction, to a height of a few feet on shore, after which it was launched, completed, towed to the site and sunk by filling the open spaces between the timbers with stones, etc. The disadvantage of this type of foundation lies in the fact that it is practically impossible to land the grillage perfectly level, owing to the great difficulty of preparing a level foundation bed. Another disadvantage lies in the

danger from scour. Further details relating to this type of foundation may be found in an article by E. K. MORSE, in Proceedings of the Engineer's Society of Western Pennsylvania, February, 1911, and in FOWLER's Sub-Aqueous Foundations.

Figure 133*b* illustrates an interesting type of shallow foundation which supports the piers of the Kingshighway viaduct, St. Louis, Mo. It consists of a reinforced-concrete box open at the bottom and closed at the top. The top has a thickness of 4 feet, while the thickness of the sides and cross-walls vary from 2½ to 3 feet. It was originally intended to found the piers on concrete piles (shown in the diagram), but in testing some of the piles already driven the soil was found to be an incompressible but perfectly plastic clay, which would not take the arch thrust with a pile foundation. By using the concrete box the clay was confined to prevent flowing action, while the large area of the sides took care of the horizontal thrust.

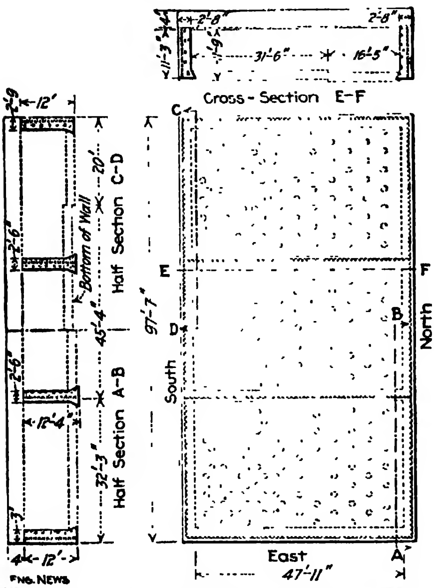


FIG. 133*b*.—Reinforced-Concrete Pier Footing, Kingshighway Viaduct, St. Louis, Mo.

ART. 134. DEFINITIONS

A bridge pier is a structure, usually composed of masonry, which is used to transmit the loads from the bridge superstructure to the foundation.

Some of the common parts of a bridge pier are the following: BRIDGE SEAT.—A block of stone or concrete resting on the top of a pier to support the pedestal or base plate. COPING.—The

top course of the pier, usually projecting beyond the other courses. BELTING COURSE.—The course immediately below the coping course. FOOTING COURSES.—Those courses at or near the bottom of the pier, which are wider than those in the main part of the pier. BODY.—The main part of the pier. STARLING.—That part of the pier below high water, the horizontal section of which lies outside of the largest rectangle that can be formed on the two sides of the pier. STARLING COPING.—The offset course at about high water which forms the top course of the starling. BATTER.—The slope of the sides and ends of the pier.

The coping course serves to protect the pier from the weather. If made of stone masonry the stone is of the best quality and cut to make small joints; if of concrete, a rich mixture is employed. The top is usually made with a surface sloping from the middle downward to the sides and is often waterproofed with some waterproofing compound, especially when of concrete. It is customary to give the coping course an offset of from 6 to 12 inches in order to prevent rainwater from dripping down the sides and ends of the pier, and also to improve the appearance of the pier.

The chief function of the belting course is to strengthen the coping offset, but it also improves the appearance of the pier. In special cases two or three belting courses are used, while at other times none are employed.

The function of the footing course is to distribute the load over a larger area than the base of the body of the pier. Unless reinforced, the slope of the footing should not be over 30 degrees with the vertical; where reinforced, the slope may be anything consistent with safe stresses in the steel and concrete as determined when considering the projecting footing courses to act as a cantilever beam.

As explained in Art. 135, the function of the starling is to pass the water with the least possible disturbance, for then there will be the least pressure against the pier due to current, ice and drift, less danger to navigation from eddies and less danger from underscouring.

ART. 135. FORM, DIMENSIONS AND QUANTITIES

The two primary requirements of bridge piers are: first, to transmit the load from the superstructure to the foundation; and, second, to disturb the natural movement of the water as little as possible. Naturally, a minimum capitalized cost should also be sought. As the load from the superstructure is applied on the pier at two points, at a distance apart equal to the width of the trusses or girders center to center, the most economical way of satisfying the first requirement is the employment of two cylinders, one under each load, as described in Art. 142. On the other hand, the second requirement is best served with a form resembling a ship, modified to increase the stability of the pier against floating ice, débris, etc., and to make the construction cheaper. The shape generally used is that of a rectangle with triangles or segments of circles at both upstream and downstream ends, or at only the upstream end. The advantage of having starlings at both ends is that the foundation becomes symmetrical with the loads, thus avoiding an uneven distribution of pressure on the foundation bed; eddying on the downstream end of the pier is also reduced. Starlings are necessary only below high water.

The triangular nose, usually made with a 90-degree angle at the vertex, has the advantage over the curved nose in cheapness of construction, but experiments show that it offers more resistance to the passage of water. Experiments made by CRESY indicate the value of different shapes of piers in passing the water to be in the following order: first, elliptical horizontal sections; second, rectangular body with starlings formed by two circular arcs, tangent to the sides and described on the sides of an equilateral triangle; third, rectangular body with triangular starlings, the angle at the nose being 60 degrees; fourth, rectangular body with semi-circular starlings; fifth, rectangular body with triangular starlings, the angle at the nose being 90 degrees; and, sixth, rectangular body without starlings.

Tests of small models of piers reported in the Transactions of the American Society of Civil Engineers, vol. 82, page 334, give the following relative efficiencies for piers with noses and

tails of the following forms: 0.923 for circular, 0.916 for 45-degree triangular, 0.893 for 90-degree triangular and 0.861 for square. The best efficiencies were obtained for sections approximating the ellipse, an efficiency of 0.939 being obtained for a pier with a nose composed of segments of a circle having a radius of three times the thickness of the pier and a tail with a reverse curve like a fishtail, the length of the tail being twice the thickness of the pier.

The above paper has a valuable discussion on the heading up of the water in a stream due to the presence of piers. Those forms which pass the current best lack strength and massiveness in their starlings. Where used in swift streams filled with ice in winter, the starlings are heavily reinforced with old rails or structural shapes. For an example of such reinforcement the reader is referred to Art. 137.

Where segments of circles are used, the curves are tangent to the sides of the pier and have radii somewhat greater than half the thickness of the pier to give a pointed end. A value used on many piers and recommended by G. S. Morison is three-quarters the width of the pier. Above high water the ends of the pier may be made square, but a much better appearance is secured when a semi-circular form is used. A combination of the straight and circular nose is sometimes adopted and is illustrated in Art. 138. Where the pier extends a considerable distance above high water, it is customary to reduce the section somewhat above that elevation. More complete details of this are given in Art. 137.

Dimensions of Bridge Piers.—The dimensions of ordinary bridge piers depend upon the load to be supported, class of superstructure, height of pier, type of foundation and magnitude of lateral forces to be resisted.

The dimensions of the top of the pier depend on the distance between trusses or girders, plus a certain amount necessary to prevent the load from the pedestals approaching too closely the edges of the pier, under the coping. Greiner specifies[1] that the width shall not be less than 4 feet, nor less than that

[1] General Specifications for Bridges, Part III, by J. E. Greiner.

required for the bearings of the superstructure plus 1 foot, nor less than that required to give the required stability. The question of stability is discussed in Art. 140. He also specifies

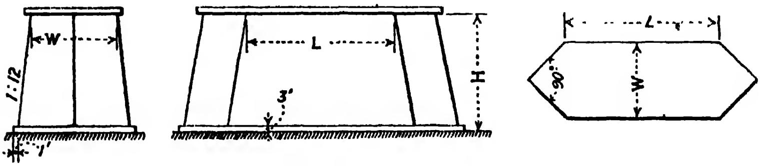


FIG. 135a.—Outline of Standard Concrete Pier.

that the length under the coping shall not be less than the distance out to out of superstructure bearings plus one and one-quarter times the width of the pier.

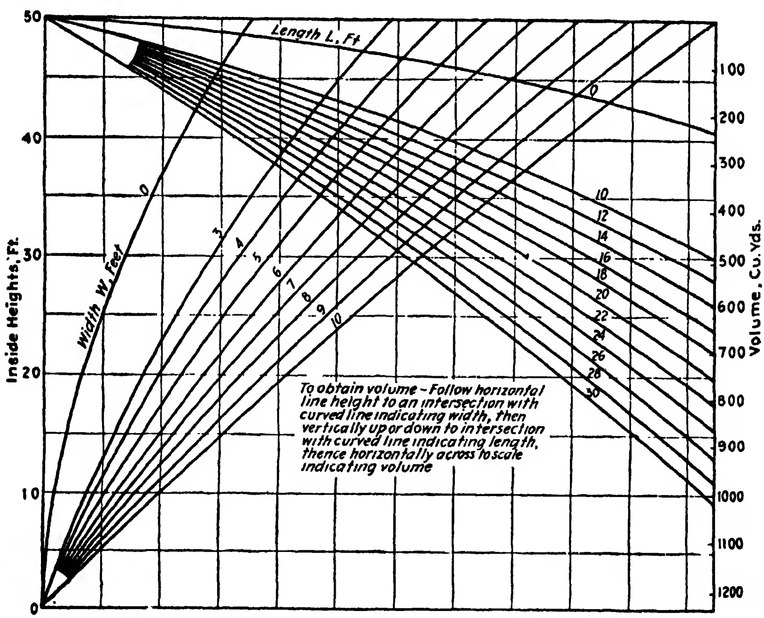


FIG. 135b.—Diagram for Cubature of Concrete Piers.

For electric-railway bridges C. C. SCHNEIDER, in an article in the Street Railway Journal, Sept, 15, 1906, specifies that the thickness of the pier under the coping should not be less than 4 feet. "The usual practice is to have the masonry on top

(under coping) project 3 inches in the direction of the thickness of the pier and at least 6 inches in the direction of the length of the pier beyond the edges of the base plate." For piers supporting two spans of approximately the same length, Table No. 135*a*,* taken from the article just noted, gives the approximate minimum dimensions for electric-railway bridge piers.

Figure 135*a* shows the standard form of pier for the Harriman Lines, while Tables Nos. 135*b* and *c*, respectively, give the

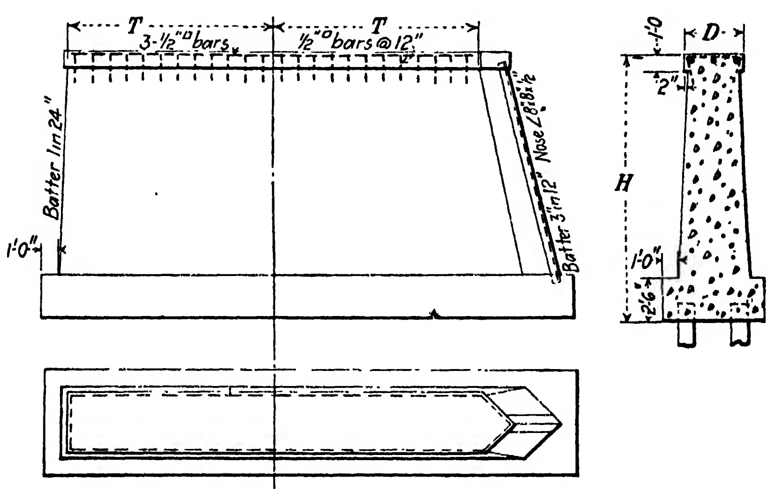


FIG. 135*c*.—Standard Highway Bridge Pier.

lengths and widths under copings for various types of superstructures. Figure 135*b* gives the volume of masonry in this type of pier for various heights, lengths and widths.

Figure 135*c* shows the type of pier used for country highway bridges by the Iowa Highway Commission. In general, piers with $D = 3$ feet are used for beam spans, concrete slabs and concrete deck girders, $D = 3\frac{1}{2}$ feet for ordinary steel spans and larger values of D for long steel spans. For 18-foot roadways the values of T are 10 feet for beam spans, 11 feet for concrete slab spans, 9 feet for concrete deck-girder spans and 12 feet for steel spans.

The quantities of concrete in cubic yards for $T = 12$ feet are as follows:

H in ft.	10.5	12.5	14.5	16.5	18.5	20.5	22.5	24.5	26.5	28.5	30.5	32.5	34.5	36.5
$D = 3$ ft.	36.6	43.9	52.0	60.3	69.1	78.5	88.4	98.9	110.2	121.8	134.0	146.8	160.2	174.3
$D = 4$ ft.	47.4	57.2	67.2	77.7	88.8	100.4	112.7	125.9	139.4	153.4	168.1	183.5	199.6	216.7
$D = 5$ ft.	58.5	70.5	82.7	95.4	108.8	122.8	137.5	153.1	169.0	185.6	202.9	220.9	239.5	259.4

The coping course usually has a thickness varying from 1 to 2½ feet, and an offset depending on the thickness. When concrete is used, GREINER specifies that "copings shall not have a less depth than 1 foot nor less than one-sixth of the thickness of the stem measured under coping. They shall project over the faces of the stem to an extent equal to about one-third their depth. This projection shall be neatly molded on the bottom and chamfered on the top and have all corners rounded." COOPER specifies that [1]"the coping shall extend at least 3 inches all around, but not more than one-third of its thickness." This specification is for highway and electric-railway bridges. The specifications for the Harriman Lines call for a 4-inch projection of coping for concrete piers and for masonry piers 10 feet and under in height, and a 6-inch projection for masonry piers over 10 feet high. In the Thebes bridge piers (Fig. 137*e*) the thickness of the stone-masonry coping is 27 inches and the projection 24 inches. The belting course not only improves the appearance of the pier but helps to secure a greater projection of the coping course. Its dimensions and form vary.

As noted in a previous article, a single or double belting may be employed, or the same may be dispensed with altogether. When used, it is usually made of about the same or somewhat less thickness as the coping course and its projection beyond the stem of the pier is closely equal to the projection of the coping course beyond the belting course. As to whether a double belting course is preferable to a single one in any given case will

[1] General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges, by THEODORE COOPER.

depend on the desired total offset of the coping course with reference to the stem of the pier. In the following articles a number of piers are illustrated which show clearly their belting courses.

TABLE 135*a*.—APPROXIMATE MINIMUM DIMENSIONS OF ELECTRIC-RAILWAY BRIDGE PIERS

Span	Thickness of pier under coping					
	Class A		Class B		Class C	
	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.
25	4-0	4-0	4-0	4-0	4-0	4-0
50	4-0	5-3	4-0	4-0	4-0	4-0
75	4-6	6-0	4-0	4-6	4-0	4-0
100	5-0	6-6	4-0	5-0	4-0	4-0
125	5-4	7-0	4-0	5-4	4-0	4-4
150	5-8	7-6	4-3	5-8	4-0	4-8
175	6-0	8-0	4-6	6-0	4-0	5-0
200	6-4	8-6	4-9	6-4	4-0	5-4
250	7-0	9-6	5-3	7-0	4-6	6-0
300	7-8	10-6	5-9	7-8	4-10	6-6
350	8-4	11-4	6-2	8-4	5-2	7-0
400	9-0	12-0	6-6	9-0	5-6	7-6

Span	Length of pier under coping = distance center to center of trusses + figures below					
	Class A		Class B		Class C	
	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.
50	3-6	4-0	3-6	3-6	3-6	3-6
100	4-0	5-0	3-6	4-0	3-6	3-6
150	4-6	5-6	4-0	4-6	3-6	4-0
200	5-0	6-0	4-0	5-0	3-6	4-6
250	5-0	6-6	4-6	5-0	4-0	4-6
300	5-6	7-0	4-6	5-6	4-0	5-0
350	6-0	7-6	4-6	6-0	4-6	5-0
400	6-0	7-6	5-0	6-0	4-6	5-6

Note: All values are expressed in feet and inches. S. T. = single track; D. T. = double track. Class A, heavy traffic; Class B, medium traffic; Class C, light traffic.

TABLE 135b.—LENGTH UNDER COPING OF CONCRETE BRIDGE PIERS HARRIMAN LINES' STANDARD, 1906

	Deck plate girders									Through riveted trusses				
Span	20	30	40	50	60	70	80	90	100	100	110	125	140	150
Length	8–4	9–2	9–0	9–2	10–0	11–0	11–0	12–2	12–2	20–0	20–0	20–0	20–8	20–8

	Through pin trusses				Through plate girders							
Span	150	160	180	200	30	40	50	60	70	80	90	100
Length	21–2	21–2	21–4	21–4	16–8	17–10	18–2	19–0	19–10	19–6	19–8	19–10

Note: Length of pier to correspond to length given in table for the longer span. All dimensions are expressed in feet and inches.

The sides of the body or stem of the pier are invariably given a batter of either 1 in 24 or 1 in 12. Above high water the ends are also given this batter. The former value is more commonly used for high piers and the latter for low piers. Either gives a pier of good appearance and will usually furnish adequate stability and a base of sufficient size.

The footing courses serve to transfer the load from the body of the pier to the foundation and for this reason they are given a larger horizontal section than the base of the body of the pier.

GREINER specifies the following in regard to their dimensions: "The upper surface of the upper footing course shall not project more than 1 foot beyond any face of the stem . . . The depth of any footing course shall not be less than 2 feet and the courses may be stepped off at an angle of about 30 degrees with the vertical or have a uniform batter of the same amount. When constructed on pile foundations, the footings shall encase the piles to a depth of at least 6 inches, and the distance from the center of any pile to the outside face of the footing shall not be less than 1½ feet."

TABLE 135c.—WIDTH UNDER COPING OF CONCRETE BRIDGE PIERS—HARRIMAN LINES' STANDARD, 1906

		Deck girders								Through riveted							Through pin		
	Span in ft.	20	30	40	50	60	70	80	90	100	100 (Through plate girders)	110	125	140	150	150	160	180	200
Deck plate girders	20	2- 4																	
	30	3- 1	3-10																
	40	3- 7	4- 4	4-10															
	50	3- 7	4- 4	4-10	4-10														
	60	3- 1	3-10	4- 4	4- 4	3-10													
	70	3- 2	3-11	4- 5	4- 5	3-11	4- 0												
	80	3- 8	4- 5	4-11	4-11	4- 5	4- 6	5-0											
	90	3- 9	4- 6	5- 0	5- 0	4- 6	4- 7	5-1	5- 2										
	100	3-10	4- 7	5- 1	5- 1	4- 7	4- 8	5-2	5- 3	5- 4									
Through riveted trusses	100	4- 0	4- 9	5- 3	5- 3	4- 9	4-10	5-4	5- 5	5- 6	5- 8								
	110	4- 1	4-10	5- 4	5- 4	4-10	4-11	5-5	5- 6	5- 7	5- 9	5-10							
	125	4- 2	4-11	5- 5	5- 5	4-11	5- 0	5-6	5- 7	5- 8	5-10	5-11	6- 0						
	140	4- 4	5- 1	5- 7	5- 7	5- 1	5- 2	5-8	5- 9	5-10	6- 0	6- 1	6- 2	6- 4					
	150	4- 5	5- 2	5- 8	5- 8	5- 2	5- 3	5-9	5-11	5-11	6- 1	6- 2	6- 3	6- 5	6- 6				
Through pin trusses	150	4- 2	4-11	5- 5	5- 5	4-11	5- 0	5-6	5- 7	5- 8	5-10	5-11	6- 0	6- 2	6- 3	6- 0			
	160	4- 3	5- 0	5- 6	5- 6	5- 0	5- 1	5-7	5- 8	5- 9	5-11	6- 0	6- 1	6- 3	6- 4	6- 1	6- 2		
	180	4- 5	5- 2	5- 8	5- 8	5- 2	5- 3	5-9	5-10	5-11	6- 1	6- 2	6- 3	6- 5	6- 6	6- 3	6- 4	6-6	
	200	4- 8	5- 5	5-11	5-11	5- 5	5- 6	6-0	6- 1	6- 2	6- 4	6- 5	6- 6	6- 8	6- 9	6- 6	6- 7	6-9	7-0

Through plate girders								
Span	30	40	50	60	70	80	90	100
30	3-10							
40	4- 0	4- 2						
50	4- 0	4- 2	4- 2					
60	4- 0	4- 2	4- 2	4- 2				
70	4- 0	4- 2	4- 2	4- 2	4- 2			
80	4- 5	4- 7	4- 7	4- 7	4- 7	5-0		
90	4- 6	4- 8	4- 8	4- 8	4- 8	5-1	5-2	
100	4- 9	4-11	4-11	4-11	4-11	5-4	5-5	5-8

All dimensions are expressed in feet and inches.

ART. 136. MATERIALS AND CONSTRUCTION

Previous to about 1880 it was the universal rule to build piers entirely of stone masonry, while at the present time most piers are built either entirely of concrete or of a concrete hearting and stone facing. Three conditions have brought about this change: first, the decrease in the cost of cement; second, the increase in the strength and the greater reliability of cement and concrete; and, third, the increased cost of cut stone, due to the labor factor.

Among the earliest of the all-concrete piers in this country were those used for a bridge across the Medina River, 18½ miles west of San Antonio, Tex., built in 1881. In Nova Scotia they were first used in 1883. In both of these instances concrete was used because of the absence of good stone in the vicinity and the high cost of transportation. In Europe the all-concrete pier was used somewhat earlier than the above dates. For some years after its introduction the development of the all-concrete pier was slow. In an address delivered in 1899, G. S. MORISON said: [1]"Prejudices have been raised against it (concrete) through inferior work done in this country when it was first introduced, but it is within the limits of possibilities that an artificial stone can be made in this way which will be as good and as durable as the natural stones which are commonly used; when this is accomplished the advantages of a truly monolithic construction will make concrete the best building material, and, except for the facings of monumental works, where nothing can take the place of the finest stone from nature's laboratory, it may be universally used."

Considering the stone-masonry pier as exemplified in many large bridges built by G. S. MORISON, the facing courses are mostly limestone ashlar, with granite ashlar for the upstream nose stones for all courses between high and low water. The backing is composed of limestone rubble, in some cases with, and in other cases without, coursed joints. For the Bellefontaine bridge, built in 1892, it was specified that the backing stones should have the same thickness as the face stones and

[1] Engineering Record, vol. 39, page 497, Apr. 29, 1899.

that the spaces between the large stones of the backing should not occupy more than one-fifth of the volume of the pier inside the face stones, and that these spaces should be filled with good rubble masonry.

The piers for the Merchant's bridge across the Mississippi River at St. Louis, built in 1889, were among the early large piers to have concrete backing. Here the coping course, the three courses below this, and the starling coping course were all

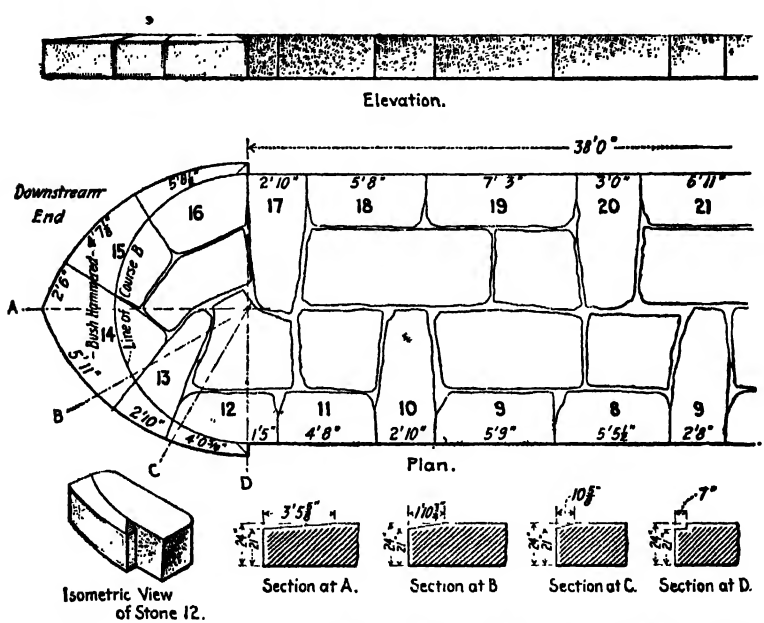


FIG. 136*a*.—Starling Coping Course for Pier IV, Merchant's Bridge, St. Louis.

of stone masonry, the remainder of the backing being concrete. Figure 136*a* shows the details of the stone masonry for the starling coping for piers I and IV.

For complete and up-to-date specifications for stone masonry the reader is referred to the Manual of the American Railway Engineering Association.

The advantage of concrete over stone masonry lies in its lesser cost. Although its compressive strength is somewhat less

than that of first-class stone masonry, yet, on account of its monolithic character, most engineers agree that it is the more suitable material, except possibly for the facing of the pier. Mixtures of 1-2½-5 or 1-3-6 proportions are usually adopted for the hearting, and a richer mixture for the coping course.

There are some advantages, however, in using a facing of stone masonry, among these being the saving in the expense of forms, the more rapid rate of construction possible, the more attractive appearance of the pier and the elimination of surface cracks. These surface cracks, almost always present in plain concrete piers, are due to the expansion and contraction, caused by temperature changes, of the outer layer of concrete.

Where a stone-masonry facing and concrete backing are used for piers bearing very heavy loads, the facing stones should be tied in with rods, as shown in Fig. 137*c*. Where the all-concrete pier is used, it is advisable to place reinforcing rods near the surface. This reinforcement will prevent the occurrence of, or at least decrease the size of, the cracks noted above, and will also add an element of safety by taking any tensile stresses in the concrete. Reinforcement in horizontal planes under the coping and above the bottom of the footing serves to carry the loads more uniformly into the pier and foundation.

GREINER'S Specifications state: "All faces of the stems above the footing courses, unless otherwise specified, shall have surface reinforcement for bonding the concrete composed of a network of round or deformed bars with meshes of about 1 foot vertical by 2 feet horizontal, the weight of metal being not less than 2¼ pounds for railway and 1½ pounds for other bridges for each square foot of surface reinforced. This network shall be embedded in the concrete to a depth of 2 inches, the horizontal rods being on the outside of the vertical rods and wired thereto. The vertical rods shall extend into the footings to an extent necessary for proper bond. The faces of copings shall have continuous surface reinforcement, of the same weight per square foot of surface as provided for stems . . .

"The lower footing course when on pile foundations shall have horizontal reinforcement for bonding the concrete composed of a layer of rods forming a network placed about 6 inches above the bed or placed around and between the embedded part of the piles, the weight of metal per square foot of network being not less than 3 pounds for railway and 2 pounds for other bridges. The stem shall have similar layers of horizontal reinforcement of the same weight per square foot as provided for surface reinforcement, embedded 1 foot below the coping, 1 foot above the footing course and at intermediate points at intervals not exceeding 20 feet. A similar network shall be embedded in the coping about 2 inches below its upper surface. The meshes in the horizontal layers of network shall be preferably square."

ART. 137. EXAMPLES OF SOLID PIERS

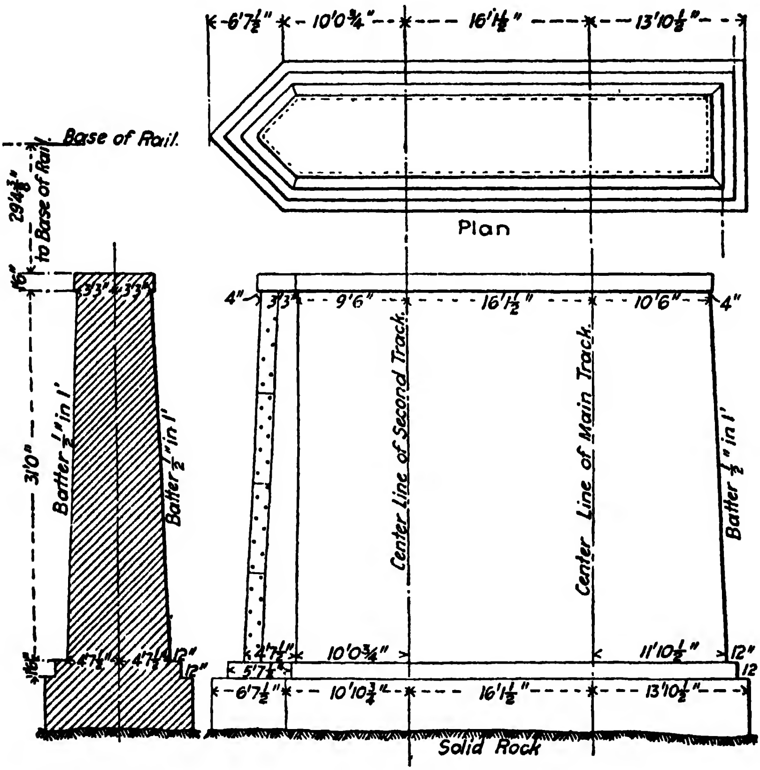
Figure 137*a* illustrates a simple form of the solid all-concrete pier used by the Western Maryland Railroad. The dimensions are given in the diagram. [1]"The upstream end of the pier is built with its sides at a 45-degree angle with its transverse axis to form a cutwater end, the nose of which extends 3 feet 3 inches beyond the corner of the pier at the lower edge of the coping. This nose was molded to a circle by inserting within the forms a strip of No. 16 iron, 9 inches wide, bent to a 6-inch radius. It is held in place by 1-inch bolts, 9 inches long, extending into the concrete. They have a welded head on the end outside the plate, and a head and a 2-inch washer on the end in the concrete."

A good example of the all-concrete pier with reinforcement near the outer surface is shown in Fig. 137*b*, which illustrates one of the piers for the Gilbertsville bridge. The bottom of the footing and the top of the coping are also reinforced.

Figure 137*c* shows the sectional elevation and plans of various courses of the part above high water of pier 3 of the Beaver Bridge of the Pittsburgh and Lake Erie Railroad. The facing

[1] Engineering Record, vol. 51, page 304, Mar. 11, 1905.

is of ashlar sandstone with 1-3-5 concrete backing. As shown in the illustrations, the facing was securely tied to the backing by 1¼-inch rods running both lengthwise and crosswise. Extra rods were used to reinforce the hearting. Contrary to the usual practice in large stone-faced piers, a stone-coping course was not



Cross-Section. Side Elevation.

FIG. 137a.—Concrete Bridge Pier, Fourth Crossing of Potomac River, Western Maryland Railroad.

used, a ring around the outside being of stone and the rest concrete. The shoe grillage of I-beams which takes the load from the superstructure and distributes it over an area of 240 square feet on the pier rests on and is supported by a 1-2-4 mixture of concrete (the darker portion in the illustration).

To waterproof the top of the pier a granolithic roof about 3 inches thick was placed over the entire top. The total load from the superstructure is 12,000 tons and the pressure on the masonry under the grillage is about 25 tons per square foot.

Figure 137*d* shows a common type where the pier is offset all around at the high-water line and has a starling-coping course projecting on both sides and ends.

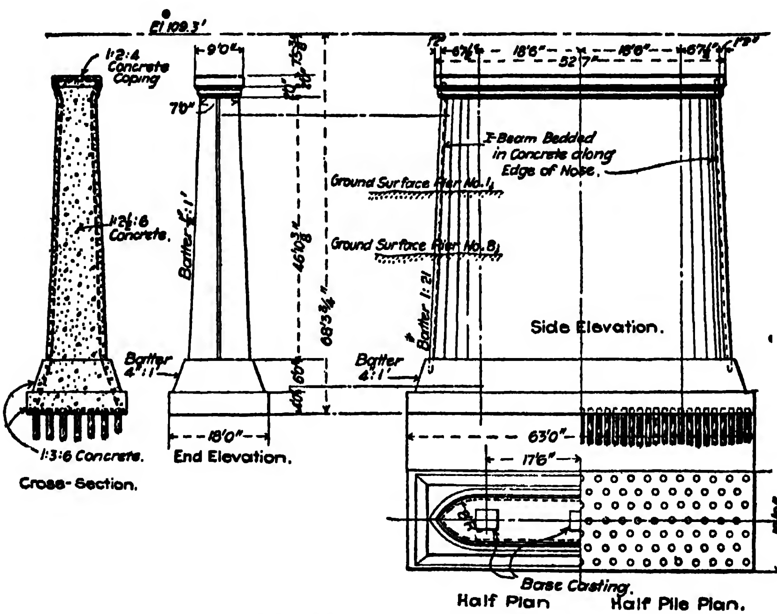


Fig. 137*b*.—General Dimensions of Piers of Illinois Central Railroad Bridge over Tennessee River, Gilbertsville, Ky.

Pier 2 of the Thebes bridge of the Illinois Central Railroad is a type of pier used in many large structures across the Mississippi and Missouri rivers. As shown in Fig. 137*e*, it is a very simple form of pier and in its simplicity lies its beauty. The sides are parallel and the ends are formed by two circular arcs meeting. Above high water the ends are semi-circular. The coping projects 2 feet beyond the pier and the projection is divided between the coping and the belting course below. The

starling coping covers the starling only. The pier has a batter of 1 in 24.

Another bridge having piers of about the same form as that just described is the McKinley bridge at St. Louis. The most notable difference between the piers of the McKinley and Thebes bridges is in the treatment of the top of the starling. As shown in Fig. 137*f* and *g*, the starling coping in the former

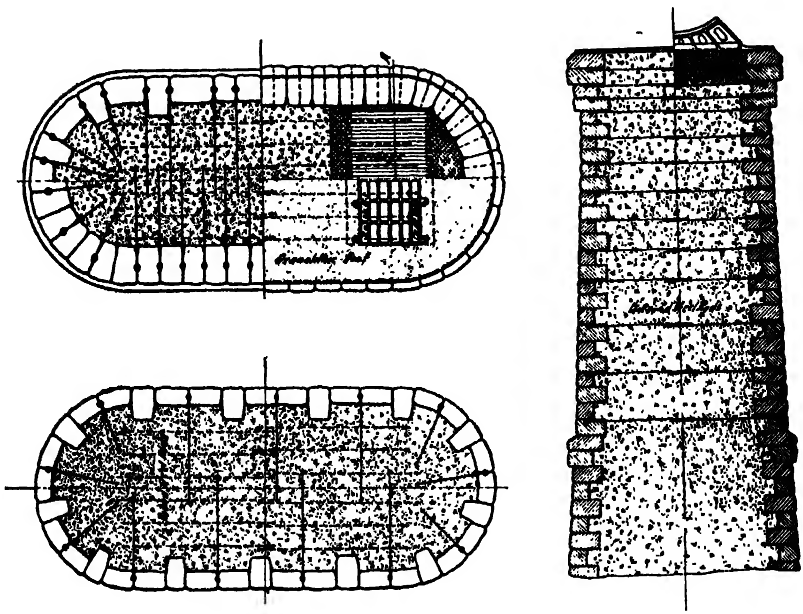


FIG. 137*c*.—Cross-section and Plans of Pier 3, Pittsburgh and Lake Erie Railway Bridge over Ohio River, Beaver, Pa.

bridge is dispensed with and the top of the starling finished with a conical surface.

For the McKinley bridge piers the facing is of limestone, with the exception of the bridge seats and the upstream nose stones above the river bed, which are of granite. The hearting is of concrete, with the exception of the three courses below the coping, which are backed with limestone masonry. [1]"The

[1] Engineering News, vol. 63, page 9, Jan. 6, 1910.

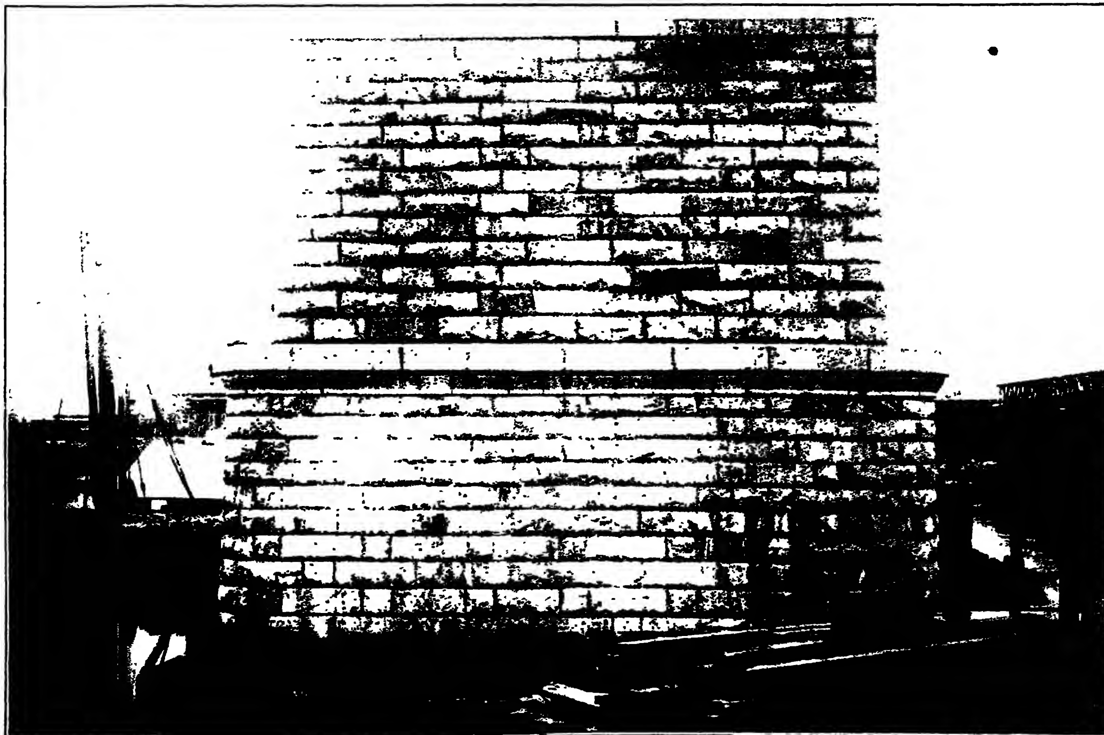


FIG. 137*d*.—Pier of Gray's Ferry Bridge over Schuylkill River, Philadelphia, Pa. July 11, 1898.

(*Facing p.* 434.)

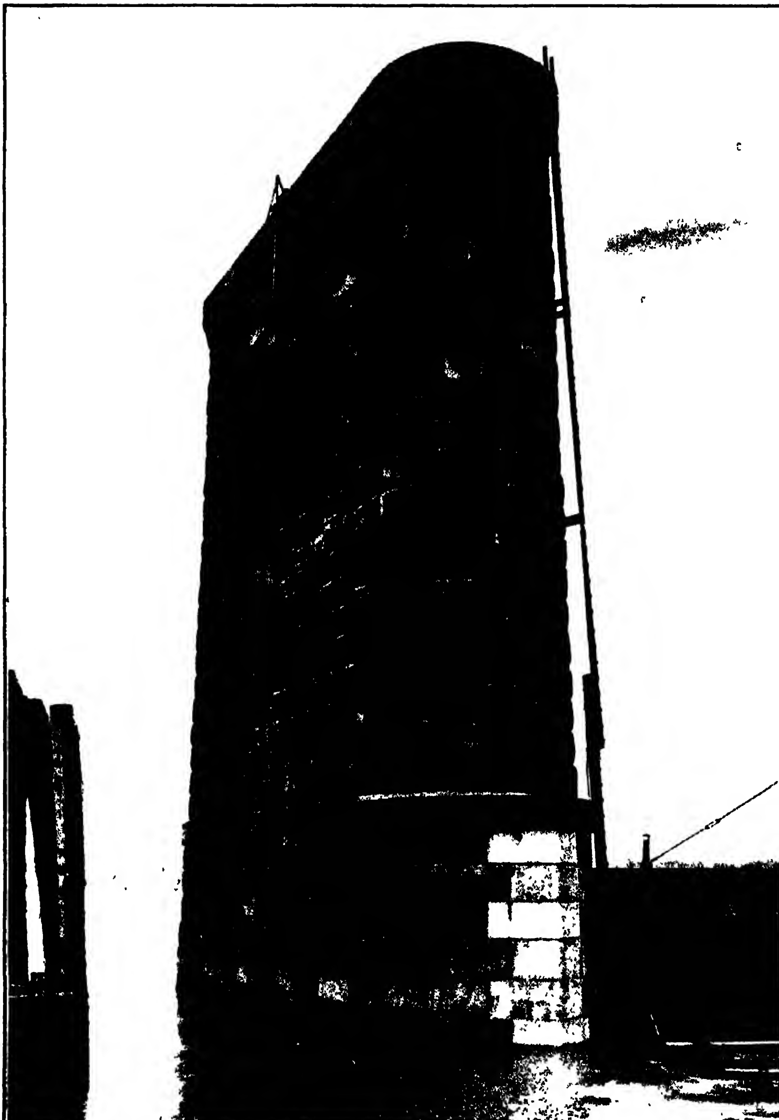


FIG. 137e.—Pier 2 of Cantilever Bridge over the Mississippi River at Thebes, Ill.
Designed by Noble and Modjeski. April 1, 1905.

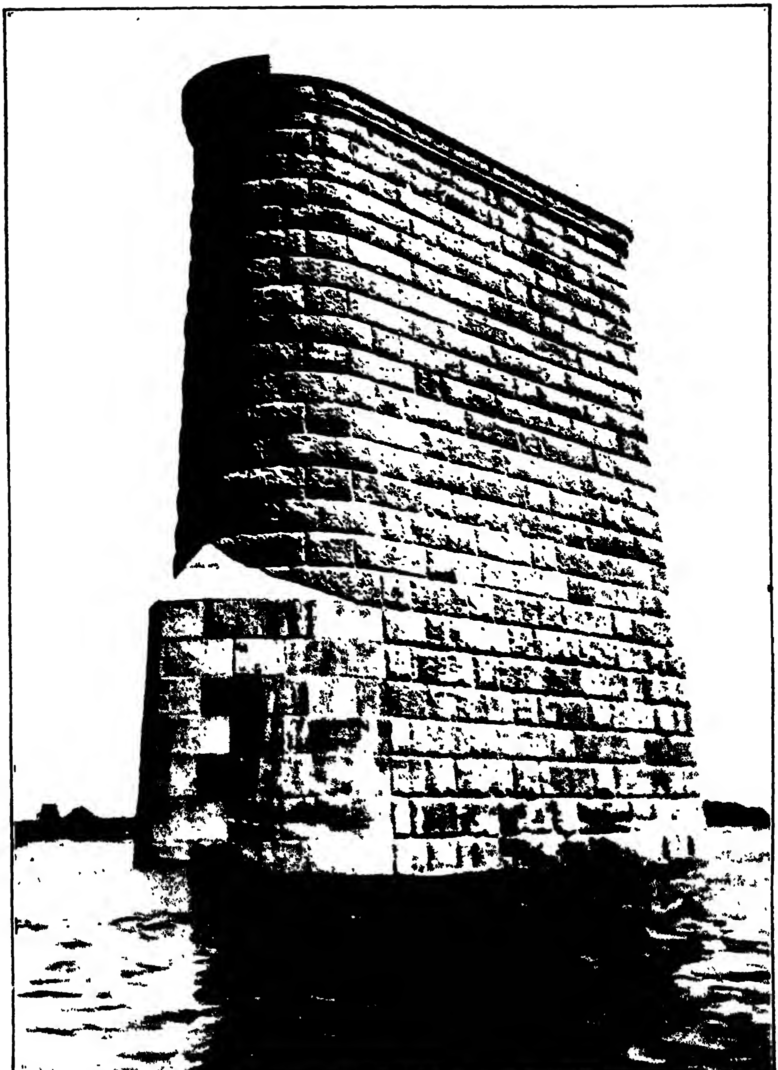


FIG. 137*f*.—Pier 3 of McKinley Bridge over the Mississippi River at St. Louis, Mo., Showing Starling with Conical Top. May 16, 1909.

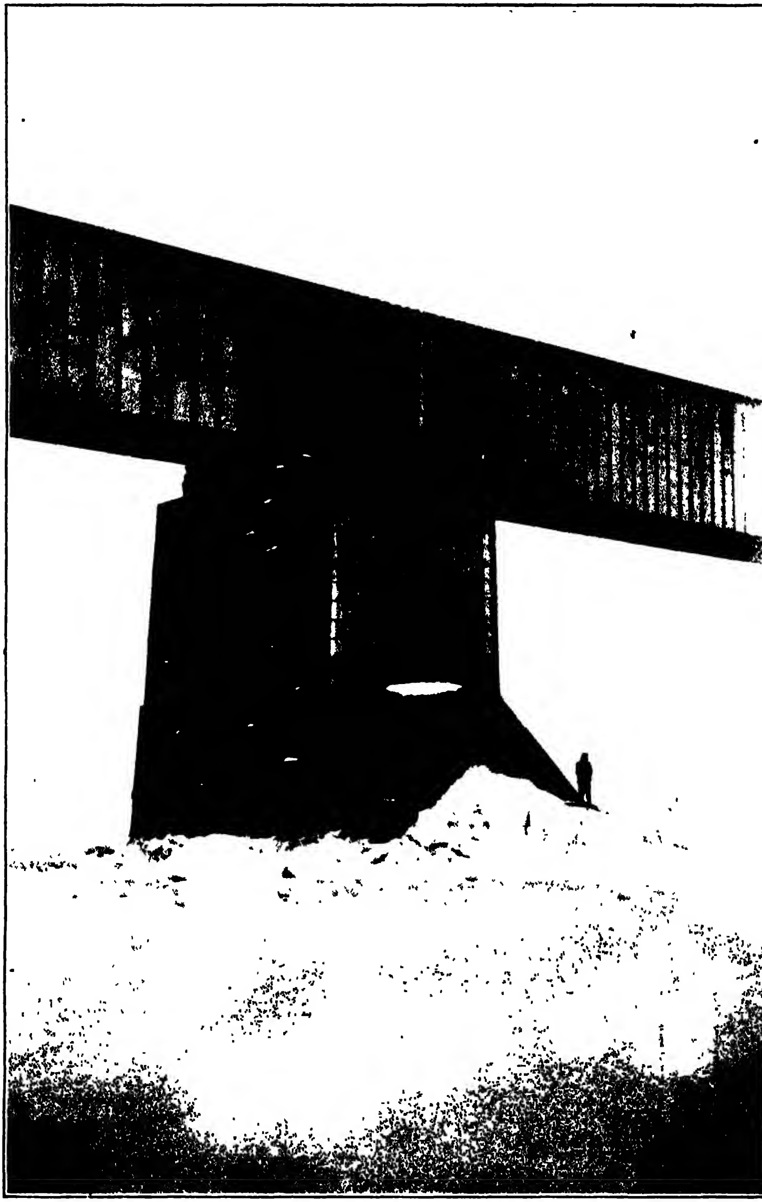


FIG. 137*i*.—A Pier of the Victoria, or Grand Trunk Railroad Bridge over the St. Lawrence River at Montreal, Ont. Built in 1858. The nose of the ice-breaker has an inclination of about 43 degrees, and is protected by iron plates. See Engineering Record, vol. 38, pages 444 and 466, Oct. 22 and 29, 1898.

(*Facing* p. 435.)

curved surfaces of the upstream starlings are close pointed to ¼-inch projection. The exposed surfaces of the main copings and the projecting bottom beds of the belting courses are planed. A 4-inch draft line is cut along the lower edges of the belting courses and on each side of the vertical angles of the down-

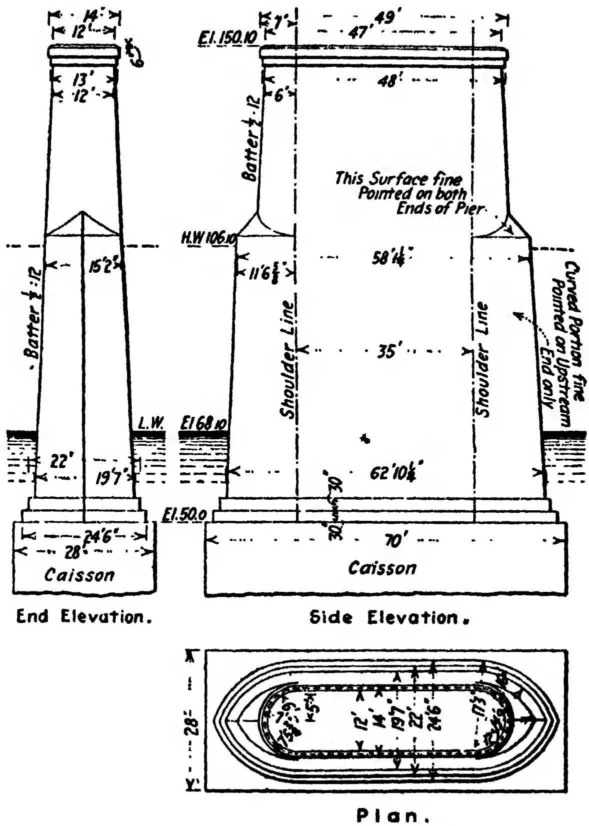


FIG. 137g.—General Dimensions of Pier 3 of the McKinley Bridge.

stream starlings. All other stones are quarry-faced, with projections not exceeding 3 inches."

The piers of the McKinley bridge were designed by RALPH MODJESKI, and those of the Thebes bridge by ALFRED NOBLE and RALPH MODJESKI. The piers for both of these bridges

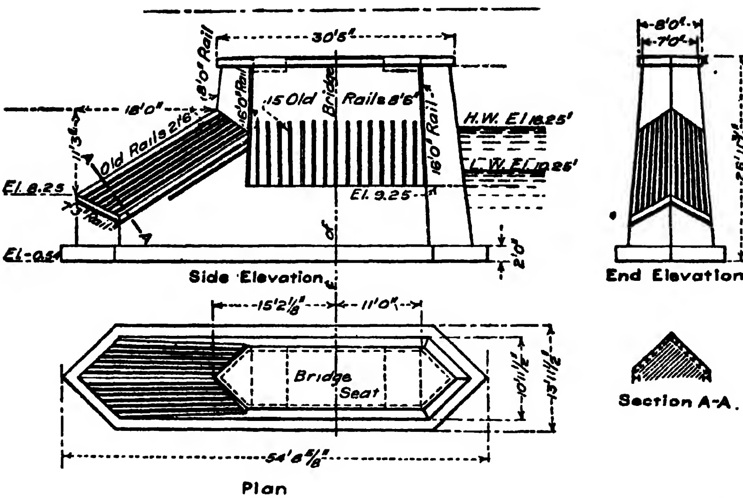


FIG. 137*h*.—Pier with Ice-Breaking Cutwater. Flag Point Bridge of Copper River and Northwestern Railway, Alaska.

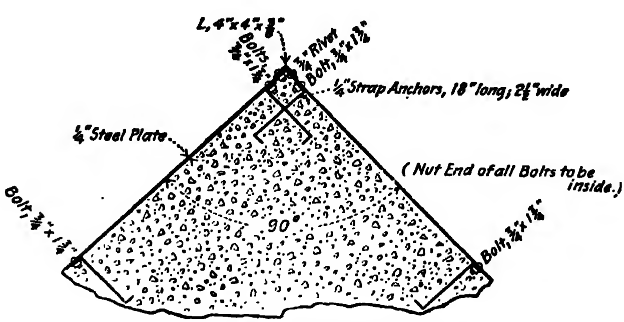


FIG. 137*j*.—Section of Steel Nose of Pier.

resemble closely the standard type designed by GEORGE S. MORISON.

Figure 137*h* illustrates the pier of a bridge across Copper River, Alaska, built to withstand very heavy ice pressure. The cutwater has a heavy slope to lift as well as to cut and divert the ice, and is heavily reinforced with old track rails. The sides are also reinforced with rails.

Figure 137*j* illustrates the steel-plate protection for the nose of a pier of the Spokane bridge of the Inland Empire System. The steel plates were 1/4 inch thick and 5 feet 8 1/2 inches wide on each side of the vertex and extended from the river bottom to above high water. They were anchored to the pier by 1/4-inch Z-shaped straps 18 inches long and spaced 18 inches apart, staggered on the nose. At the vertex the plates were reinforced with a 4- by 4- by 3/8-inch angle.

In Arts. 86 and 91 are more illustrations of solid piers.

ART. 138. EXAMPLES OF HOLLOW PIERS

In the solid bridge pier a considerable part of the hearting near the top of the pier and between the pedestal bearings takes but little load. In other words, the pier acts more or less like a double-cylinder pier, the part directly under the bearings acting somewhat as independent legs to carry the load, the remainder acting chiefly as a bracing system. For this reason a considerable amount of concrete may be saved with but small loss of strength by making the pier more or less hollow. However, when this is done the remaining concrete should be well reinforced. It is not advisable in all cases to dispense with any of the filling, for massiveness or weight tends to reduce vibration.

The hollow pier is a compromise between the solid and the cylinder pier; it is less expensive than the former but has somewhat less stability and rigidity; it is more expensive than the latter but is far more stable and makes a more attractive substructure.

The river piers of the Municipal bridge across the Mississippi River at St. Louis, Mo., illustrate the hollow type of pier. As

shown in Figs. 138*a* and *b*, the part above high water consists of a tall battered shaft with a large hollow interior space, virtually forming two independent shafts braced together with a well-reinforced arch at the top and walls of masonry on the

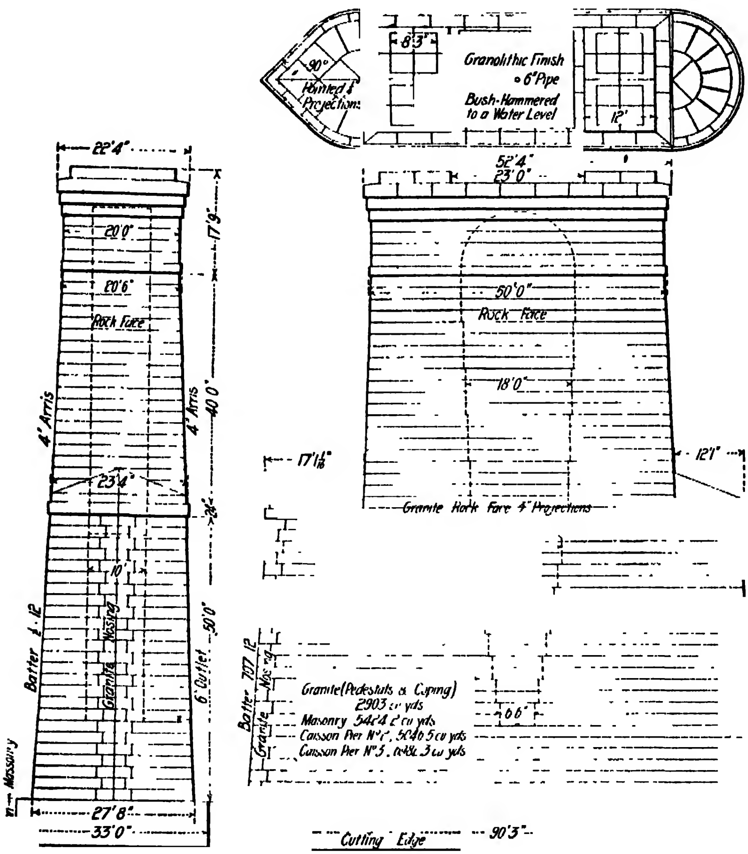


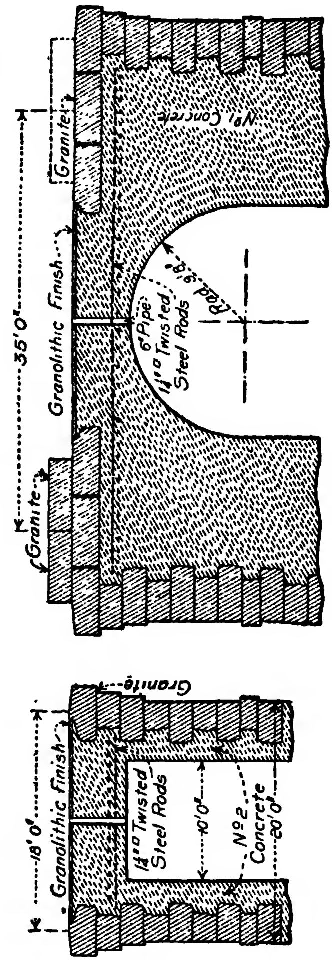
FIG. 138*a*.—Channel Piers of the Municipal Bridge over the Mississippi River, St. Louis, Mo.

sides, the latter also serving to give it the appearance of a solid pier. There is a hollow space of less size below high water. This pier is also of interest on account of the shape of cutwater which, as shown in the plan view (Fig. 138*a*), is a combination of the straight and curved types for the upstream end and

semi-circular for the downstream end. The contract price for these piers was \$9.50 per cubic yard from the top of the crib to the coping, and \$1.90 per cubic foot for coping and bridge seats.

A hollow pier resting on a pile foundation and supporting reinforced-concrete slabs is illustrated in Fig. 138*c*. The concrete for the footings was a 1-2-4 mixture, while that for the pier shaft was a 1-2½-5 mixture. In all, 186 piers of this type were used on two bridges of the Pennsylvania Railroad.

The piers of the Sparkman Street bridge, Nashville, Tenn., are shown in Fig. 138*d*. [1]"They consist of two concrete towers extending from bridge seat to footing course, and battered on all sides ½ inch to 1 foot, being braced together by a reinforced-concrete arch and corbeled coping course at the top, and by reinforced side curtain walls from the footing course up to the high-water line. The curtain walls are 2 feet thick at the top, carried down plumb on the inside, and battering with the towers on the outside. The walls are reinforced with a heavy meshed fabric placed near both inside and outside faces, this fabric extending also entirely around the towers up to the top of the curtain walls."



Sections on Center Lines of Pier

FIG. 138*b*.—Chamber in Upper Part of Piers 2 and 3. Municipal Bridge.

[1] Engineering News, vol. 26, page 576, Nov. 25, 1909.

Probably the boldest example of the hollow bridge pier is that for a bridge across the Willamette River near Portland, Ore., which is illustrated in Fig. 138*e*. The bottom 10 feet of the pier is of solid concrete, while above this it consists of a reinforced-concrete shell and reinforced-concrete columns, the latter carrying directly the loads from the superstructure. The hollow part is braced by horizontal reinforced-concrete diaphragms.

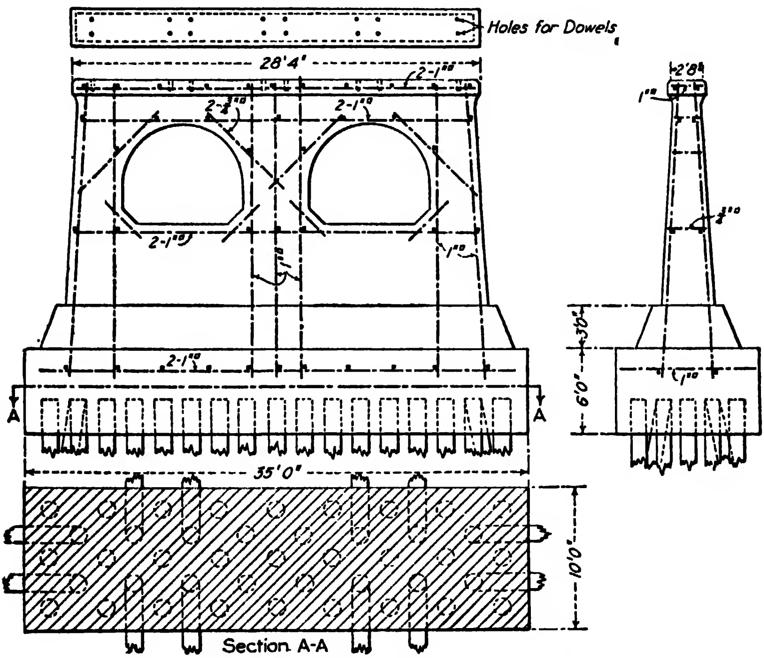


FIG. 138*c*.—Piers for Bush and Gunpowder River Bridges.

From the bottom to the top the sides are battered 1 inch to the foot.

The side walls are 18 inches thick and [1]"are reinforced as vertical slabs spanning horizontally between the columns and vertically between the horizontal diaphragms . . .

"Between the outside walls of the hollow superstructure are seven reinforced-concrete horizontal diaphragms each 2 feet

[1] Engineering Record, vol. 62, page 160, Aug. 6, 1910.

thick, which are designed as flat plates that may be loaded from above or below . . . At the center of each diaphragm is a 3-foot circular hole which permits the free passage of water between the eight stories . . . In one side of the lowest

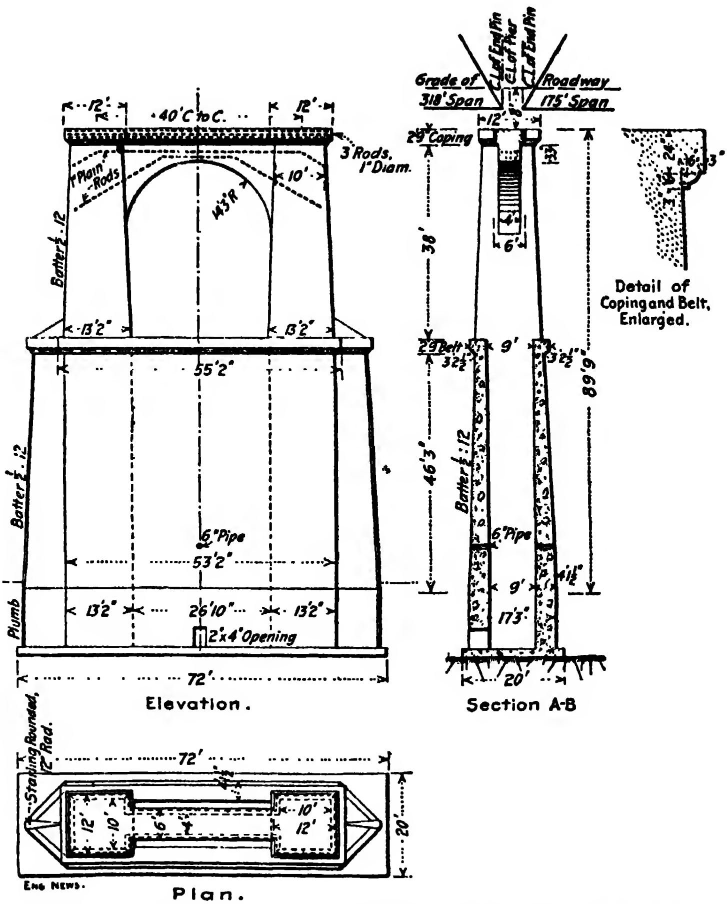
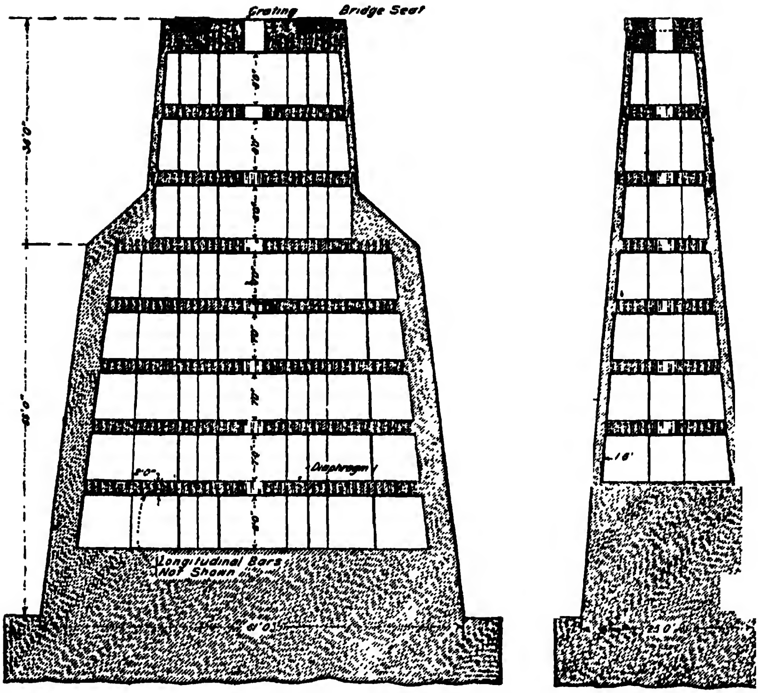
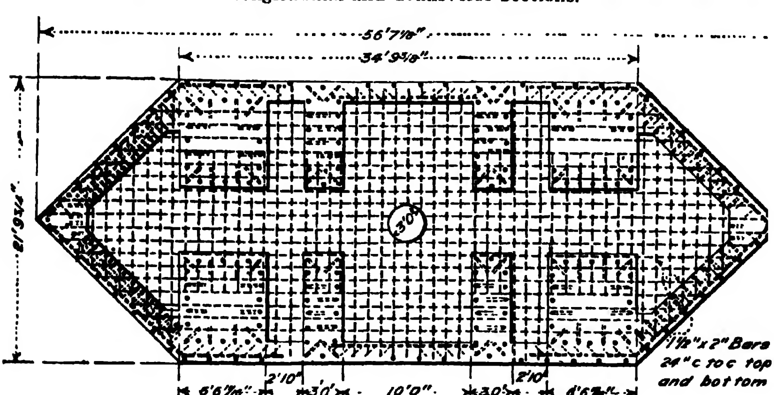


FIG. 138*d*.—Typical Channel Pier, Sparkman Street Bridge, Nashville, Tenn. Arched above and hollow below starling-coping course.

story is an opening 12 inches wide and the full height of that story, so water may rise and fall inside the pier with variations ni the stage of the river. The reinforced-concrete walls, . . . therefore, are not normally subject to a head of water, but the



Longitudinal and Transverse Sections.



Plan of Diaphragm No. 1.

FIG. 138e.—Upper Part of Tall Reinforced-Concrete Pier, Oregon-Washington Railroad and Navigation Company, Bridge over Williamette River, Portland, Ore.

design provides for any emergency that may occur by including in them reinforcement placed so that a head may be brought against the walls and diaphragms from any direction."

A pier 221 feet 9½ inches high was built as a hollow reinforced-concrete tube—virtually a chimney—for a bridge over the Dix River in Kentucky. The foundation was a 35-foot square slab, 6½ feet thick and reinforced top and bottom. The shaft was approximately 28 feet long and 18 feet wide at the bottom, with a shell thickness at the bottom of about 27 inches, this thickness decreasing toward the top. In horizontal section the pier consisted of a rectangle with semi-circular ends. The top of the pier, 22 feet long and 12 feet wide, was surmounted with a concrete slab. The shell reinforcement consisted of vertical and spiral bars. At intervals of 45 feet vertically 12- by 20-inch reinforced-concrete beams were placed to brace the walls of the shaft. Three-foot square shell ports at the top and bottom permit water to stand at the same elevation inside as outside the shaft.

For further illustrations of numerous types of piers the reader is referred to the Proceedings of the American Railway Engineering Association, vol. 18, pages 859 to 874, 1917.

Art. 139. Timber Piers

Two general types of timber piers are used, the pile and the crib, or a combination of the two. Pile piers are composed of two parts, the supporting pier and the nose. These two parts are sometimes independent structures, but more often they are combined to form a single unit. The main pier is composed of two pile bents spaced from 4 to 8 feet apart, the number of piles, and the amount and position of sway and longitudinal bracing depending on the penetration of the piles, the height of pier, superimposed load and ice and stream-flow conditions.

The type of timber pier, used by the Rock Island Lines for a bridge over the Platte River, is shown in Fig. 139*a*. This pier is composed of two rows of six creosoted piles, capped with concrete. The pier is sheathed on the outside and stiffened with

diagonal bracing. A batter pile at the nose serves as an ice breaker, the nose being protected by a bent plate spiked to the sheathing. The 24-inch concrete slab is reinforced with a network of bars near the top and bottom faces and with a stirrup system. This slab serves the double purpose of distributing the load and protecting the pier against fire.

Figure 139*b* illustrates the type of pier used in the construction of the Alaska Government Railroad. Here, four rows of

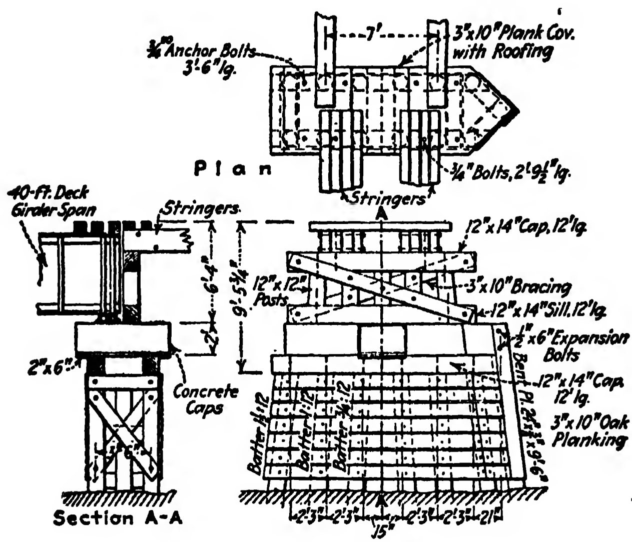


FIG. 139*a*.—Timber Pile Pier for Bridge over Platte River.

piling are used with diagonal cross- and longitudinal bracing. The outside sheathing adds stiffness to the structure and prevents lodging of ice and drift against the pier.

Crib piers, which are used where piles cannot be driven, are built of logs or of square timbers. The main body of the pier is usually rectangular and is divided into compartments by cross-ties. The logs or timbers may be halved at the corners to make a tight crib or laid one on the other without framing, thus forming a crib with openings. Where round logs are used, they should always be flattened at the ends to a bearing surface at least 4 inches wide. Each log or timber should be drift-

bolted to the one below with ⅝-inch bolts long enough to extend through the log below and about 2 inches into the second one below.

In streams having a considerable velocity or carrying ice and débris, there should be a wedge-shaped nose, and if there is danger of scouring action, the same kind of tail. The nose angle should

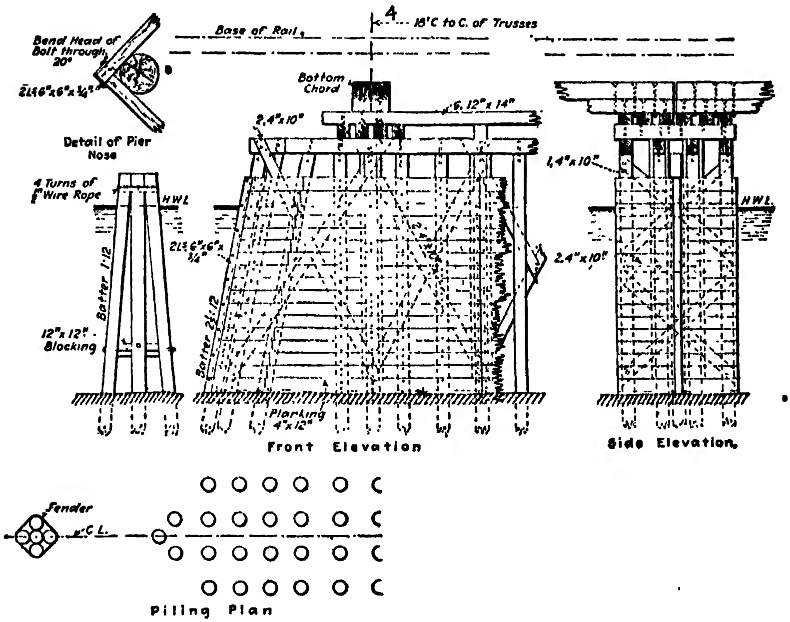


Fig. 139*b*.—Timber Crib Pier Used on Alaska Government Railroad.

be about 60 degrees and it is advisable to give the nose a slope.

If the pier is to be sunk into a soft foundation, the cross-ties should be far enough above the bottom of the crib to permit easy sinking and a floor should be placed in the plane of these ties. Cribs are filled with stone to give stability.

Art. 140. Stability of Piers

Loads.—The vertical forces to be sustained on any horizontal plane of a bridge pier are the live load, impact load, weight of superstructure and weight of pier above the plane in question.

Impact loads are usually ignored, but more generally on highway than on railroad bridge piers. For the latter, some consideration should be given to impact forces for low piers and for the upper part of high piers.

The lateral forces to be resisted by a railroad pier are tractive forces, wind on train, wind on trusses, wind on pier, river current and ice pressure. It is customary to specify a tractive force equal to two-tenths of the live load; where the bridge is a double-track structure some authorities specify a full live load on both tracks and others on one track, the latter being more general. Recent tests on the Pennsylvania Railroad with electropneumatic brakes indicate a tractive coefficient as high as 0.30. For highway bridge piers tractive forces may usually be neglected.

The wind load on train and trusses should be the same as those used in designing the superstructure, which is customarily taken at 30 pounds per square foot of exposed vertical surface of both trusses and train, or 150 pounds per linear foot of bridge for each lateral system, applied at the panel points, and 300 pounds per linear foot of train applied at a point 7 feet above the base of rail. Wind on the end of the pier may be taken at 30 pounds per square foot where the ends are without starlings and 20 pounds per square foot of vertical projection where starlings are present.

The law governing the pressure on bridge piers due to a river current is not definitely known. The formula $P = (Kwv^2)/2g$ is frequently used, in which P is the pressure in pounds per square foot of vertical projection, K a constant, v the velocity of current in feet per second, w the weight of a cubic foot of water and g the acceleration due to gravity (approximately 32.2 feet per second per second). GREINER in his General Specifications for Bridges, Part III, Substructures and Concrete Bridges, gives a value for $(Kw)/2g$ of 1.5 for flat surfaces and one-half of this for rounded surfaces, with a minimum of 150 pounds per square foot for flat surfaces subjected to freshets and 50 pounds in tidal streams, with one-half of these values for rounded ends.

Experiments show that the velocity varies with the depth approximately as the ordinates of an ellipse, the maximum being somewhat below the surface. The center of pressure is commonly assumed at one-third the distance from the water surface to the river bed. This assumption is on the safe side.

Ice exerts its greatest pressure when in the form of a field of moving ice forcing its way past the pier. In this condition the ice is more or less soft. In the specifications noted above a value of 50,000 pounds per foot of pier width for a 10-inch thickness of ice (417 pounds per square inch) is given for flat surfaces, and one-half of this value for rounded surfaces. Other thicknesses will have proportionate values. For the North Side Point bridge, Pittsburgh, Pa., the river piers, which had rounded ends, were designed to resist a horizontal ice pressure of 48,000 pounds per linear foot of width. A value used in the design of a number of large dams in this country is 47,000 pounds per linear foot of width.

Uplift is considered by many engineers in designing piers. As a result of a questionnaire by the American Railway Engineering Association in 1917 it was found that approximately two-thirds of the railroads design for uplift.

STRESS ANALYSIS.—To be stable a pier must be safe against sliding and crushing and should be free from tension on any horizontal plane and on the base. The unit sliding force is found by dividing the resultant horizontal force above the section by the area of the section.

The maximum compressive stress is given by the formula

$$f = \frac{P}{A} + \frac{Mc}{I} + \frac{M'c'}{I'},$$

where P denotes the total vertical load; A the area of the section; M and M' the moments due to forces at right angles to and parallel to the long axis of the pier, respectively; c and c' one-half the width and length of pier, respectively; and I and I' the moments of inertia of the pier section about the long and short axis, respectively. The minimum stress is found by making the last two terms of this equation negative. In

designing a pier, if on analysis the minimum stress is negative the section should be increased until a positive value is obtained.

If the section cannot take tension, the above formula does not apply where the minimum stress becomes negative. Such a case is best handled by the use of diagrams, as illustrated by Fig. 140*a*, which is applicable for the solution of rectangular footings. The maximum stress is given by the formula

$$f = \frac{KP}{A}.$$

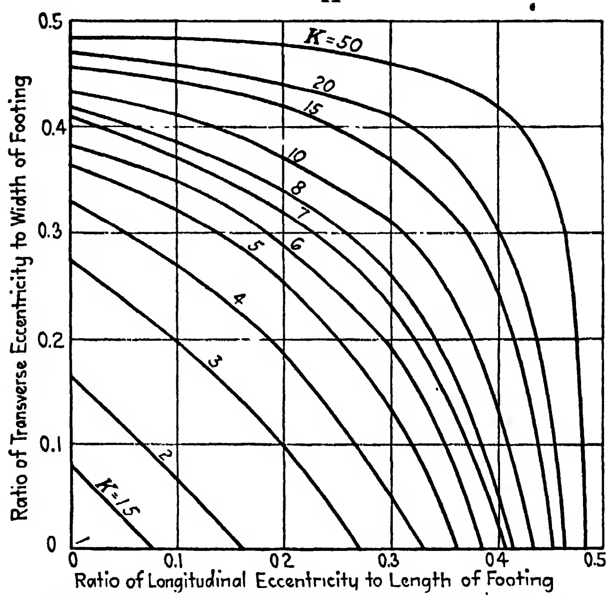


FIG. 140*a*.—Diagram of Coefficient for Eccentric Loads.

The longitudinal eccentricity is M'/P and the transverse M/P. With these known, the value of K is easily taken from the diagram. For example, with a longitudinal eccentricity of 0.3 and a transverse eccentricity of 0.4, the value of K is approximately 19.

For formulas for stresses on rectangular footings due to doubly eccentric loads the reader is referred to a valuable article by M. G. FINDLEY in the Engineering News-Record, vol. 85, page 494, Sept. 9, 1920.

The forces resisting sliding are friction of masonry on masonry for stone-masonry piers, the shearing strength of the concrete for

concrete piers and a combination of both for combination piers. For a table giving friction values for various kinds of stone masonry and for the shearing strength of concrete, see American Civil Engineers' Pocket Book. If the pier dimensions at the top accord with standard practice as outlined in Art. 135, and if the pier has the conventional batter of 1 in 12 or 1 in 24, all sections will be amply safe against sliding.

DOUGLAS[1] recommends the following allowable compressive unit-stresses in pounds per square inch: stone masonry with 1–2 portland cement mortar and joints not over ½ inch thick, granite, 700; hard limestone, 650; medium limestone and marble, 600; soft limestone and sandstone, 500; where joints are over ½ inch thick, 450 pounds for all kinds of sound building stones; for 1-2-4 concrete, 450; 1-3-6 concrete, 350; and 1-4-8 concrete, 250.

ART. 141. EXAMPLE OF PIER DESIGN

The following example, which analyzes the pressures on the foundation of pier 5 (Fig. 141*a*) of the Tennessee River bridge of the Illinois Central Railroad, is taken in part from an article by W. M. TORRANCE in Engineering News, vol. 53, page 548, May 25, 1905. Wind on pier, current and ice were not considered in the original article. The bed of the river is slightly exposed at low water.

Yardage of concrete:

Upper 2 feet of coping, 1-2-4 concrete; area in plan, 609.3 square feet; volume............	45.1 cubic yards
Lower 2 feet of coping, 1-2½-6 concrete; area in plan, 548.5 square feet; volume............	40.6 cubic yards
Shaft of pier, 1-2½-6 concrete; top area, 489.4 square feet; bottom area, 782.8 square feet; medium area, 638.7 square feet; volume by prismoidal formula (for height 56.86 feet)....	1343.2 cubic yards
Footing course, 1-3-6 concrete; top area, 883.8 square feet; bottom area, 1738 square feet; volume by end areas.....................	291.3 cubic yards
Foundation course, 1-3-6 concrete; area in plan, 2376 square feet; volume.............	352 cubic yards

[1] See American Civil Engineer's Pocket Book, page 576.

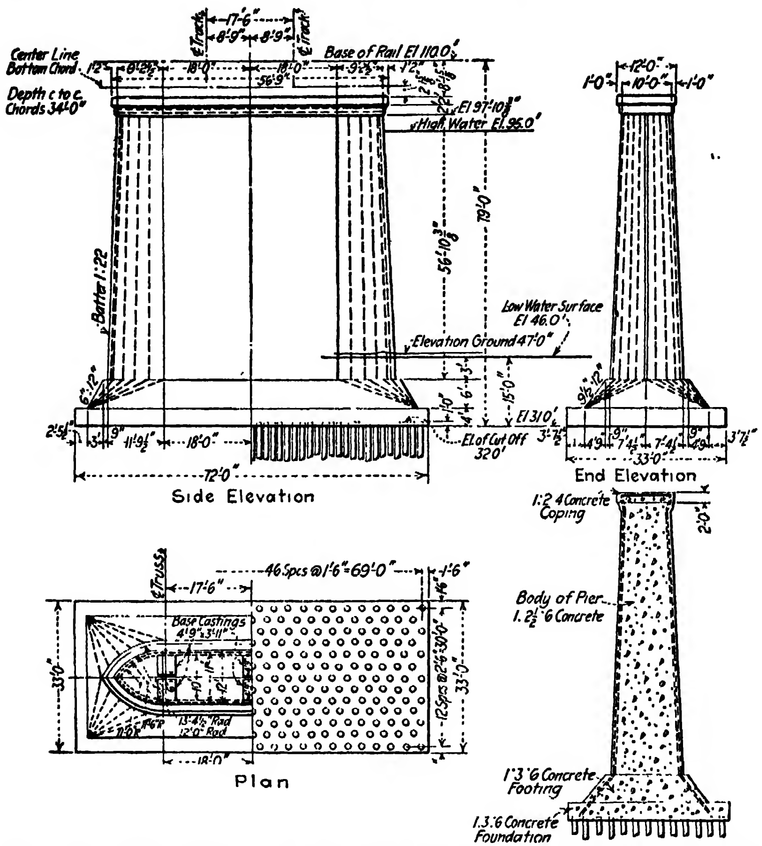


FIG. 141a.—River Pier 5, Illinois Central Railroad Bridge over Tennessee River, Gilbertsville, Ky.

Summary:

1-2-4 concrete in coping....	45.1 cubic yards	
1-2½-6 concrete in coping and shaft................	1383.8 cubic yards	
1-3-6 concrete in footing and foundation.............	643.3 cubic yards	
Total................................		2072.2 cubic yards
Weight of pier at 155 pounds per cubic foot, 2072.2 × 27 × 155..........................		8,672,000 pounds
Dead load, three trusses with ballast floor, 9162 × 300...............................		2,738,000 pounds
Live load from 300 feet of double-track train loads, 5000 × 2 × 300......................		3,000,000 pounds
Total gravity load on foundation...............		14,410,000 pounds

Using a tractive coefficient of 0.20, the

Tractive force = 3,000,000 × 0.2 = 600,000 pounds

Assuming this tractive force to act along the center line of the lower chord the

Tractive moment = 600,000 × 73 = 43,800,000 foot-pounds.

To get the moment of forces transverse to the bridge:

Moment of wind on upper lateral system, 150 × 300 × 107	4,815,000 foot-pounds
Moment of wind on lower lateral system, 150 × 300 × 73	3,285,000 foot-pounds
Moment of wind on train, 300 × 300 × 86	7,740,000 foot-pounds
Total moment from wind on superstructure	15,840,000 foot-pounds

The projection, on a vertical plane transverse to the long axis of the pier, of the part of the pier subjected to wind at low water is 724 square feet and the distance from the foundation bed to the center of gravity of this area is 43.9 feet.

Moment due to wind on pier at low water,
20 × 724 × 43.9 = 635,700 foot-pounds

Moment due to river current,
$0.75 \times 10^2 \times 588 \times 48 = 2,116,800$ foot-pounds

Moment due to ice pressure,
25,000 × 10.25 × 62.5 = 16,271,800 foot-pounds

The following computations of unit loads on base are made by (1) assuming the earth to take all the load and (2) assuming the piles to take it:

Direct load on base due to,

Weight of Superstructure

Per square foot....	2,738,000/(72 × 33) =	1152 pounds =	0.58 tons
Per pile...........	2,738,000/306 =	8950 pounds =	4.47 tons

Weight of Substructure

Per square foot....	8,672,000/(72 × 33) =	3650 pounds =	1.82 tons
Per pile...........	8,672,000/306 =	28,340 pounds =	14.17 tons

Live Load

Per square foot....	3,000,000/(72 × 33) =	1264 pounds =	0.63 tons
Per pile...........	3,000,000/306 =	9800 pounds =	4.90 tons

Uplift at High Water

Area of pier at high water, 504.6 square feet; volume of shaft of pier above high water, 52.7 cubic yards; total uplift, 1933.8 × 62.5 × 27 = 3,263,000 pounds.

Per square foot....	3,263,000/(72 × 33) =	1373 pounds =	0.69 tons
Per pile...........	3,263,000/306 =	10,660 pounds =	5.33 tons

Uplift at Low Water

Area of pier at low water, 755.6 square feet; volume of pier shaft below low water, 143.5 cubic yards; total uplift, $786.8 \times 62.5 \times 27 = 1,327,700$ pounds.

Per square foot.... $1,327,700/(72 \times 33) = 559$ pounds $= 0.28$ tons
Per pile........... $1,327,700/306 = 4340$ pounds $= 2.17$ tons

The moment of inertia of the base in bi-quadratic feet about an axis through the center of gravity and parallel with the long axis of the pier is $(72 \times 33^3)/12 = 215,600$.

The maximum and minimum pressures on the base due to tractive force are:

$(43,800,000 \times 16.5)/215,600 = \pm\ 3350$ pounds per square foot $= \pm$ 1.67 tons per square foot.

The moment of inertia of the pile tops about an axis through the center of gravity and parallel with the long axis of the pier and in units of the area of one pile top times quadratic feet is $2[24(5^2 + 10^2 + 15^2) + 23(2.5^2 + 7.5^2 + 12.5^2] = 26,860$.

The maximum and minimum loads per pile due to tractive force are $(43,800,000 \times 15)/26,860 = \pm 24,450$ pounds $= \pm 12.22$ tons.

The moment of inertia of the base in bi-quadratic feet about an axis through the center of gravity and parallel with the short axis of the pier is $(33 \times 72^3)/12 = 1,026,000$.

The maximum and minimum pressures per square foot on the base due to the following:

For wind on trusses,

$(8,100,000 \times 36)/1,026,000 = \pm 284$ pounds $= \pm 0.14$ tons.

For wind on train,

$(7,740,000 \times 36)/1,026,000 = \pm 272$ pounds $= \pm 0.14$ tons.

For wind on pier,

$(635,700 \times 36)/1,026,000 = \pm 22.3$ pounds $= \pm 0.01$ tons.

For river current and ice,

$(18,389,000 \times 36)/1,026,000 = \pm 645$ pounds $= \pm 0.31$ tons.

The moment of inertia of the pile tops about an axis through the center of gravity and parallel with the short axis of the pier, and in units of the area of one pile top times quadratic feet (neglecting moment of inertia about the gravity axis of the individual pile tops), is

$$2[7(1.5^2 + 4.5^2 + \ldots 34.5^2) + 6(3^2 + 6^2 \ldots 33^2.)] = 127,100.$$

The maximum and minimum loads per pile are as follows:

For wind on trusses,

$(8,100,000 \times 34.5)/127,100 = \pm 2189$ pounds $= \pm 1.10$ tons.

For wind on train,

$(7,740,000 \times 34.5)/127,100 = \pm 2100$ pounds $= \pm 1.05$ tons.

For wind on pier,

$(635,700 \times 34.5)/127,100 \pm = 173$ pounds $= \pm 0.087$ tons.

For river current and ice,
$(18,389,000 \times 34.5)/127,100 \quad \pm 4991$ pounds $= \pm 2.50$ tons.

It will be seen that the maximum pressure, assuming no uplift, is 5.29 tons per square foot, or 40.41 tons per pile, while with uplift the values are, respectively, 4.71 and 35.83. The minimum values show that compression always exists, although at some points it is very slight.

Summary of unit-loading on foundation:

		Tons per square feet	Tons per pile
Weight of superstructure		0.58	4.47
Weight of pier		1.82	14.17
Live load		0.63	4.90
Uplift at high water		0.69	5.33
Uplift at low water		0.28	2.17
Tractive force		1.67	12.22
Wind on trusses		0.14	1.10
Wind on train		0.14	1.05
Wind on pier		0.01	0.09
River current and ice		0.31	2.50
Assuming no uplift	Maximum	5.29	40.41
	Minimum	0.77	6.67
Assuming full uplift	Maximum	4.71	35.83
	Minimum	0.08	1.34

Regarding the effect of uplift, in a case like this, where water is more or less free to get under the pier, there is no question of its action. On the other hand, it cannot act with full hydrostatic pressure on account of the presence of gravel and of the pile tops bearing against the pier.

In this pier, where the top is but a slight distance above high water, wind on pier cannot act simultaneously with ice and current, or, at least, that which acts may be neglected. In computing the minimum pressure the live load is included as the negative values due to tractive force and wind on train overbalanced the positive value due to direct pressure. In finding the maximum pressure by considering uplift, the condi-

tions obtaining at low water were used, since these give a greater value than for high water. In getting the minimum values with uplift, high water was used.

The above maximum stress may be checked by the use of Fig. 140*a*. Not considering uplift, the transverse eccentricity is 3.04 feet and the longitudinal eccentricity 2.35 feet. With the two ratios of 0.092 and 0.33, the value of K is found to be approximately 1.75. The value of P/A for dead and live loads is 3.03 tons per square foot, hence the maximum stress from the formula $f = KP/A$ is 5.30 tons per square foot.

In studying the horizontal section of the pier the same method is to be followed as in obtaining the pressure on the base, except that uplift will be omitted.

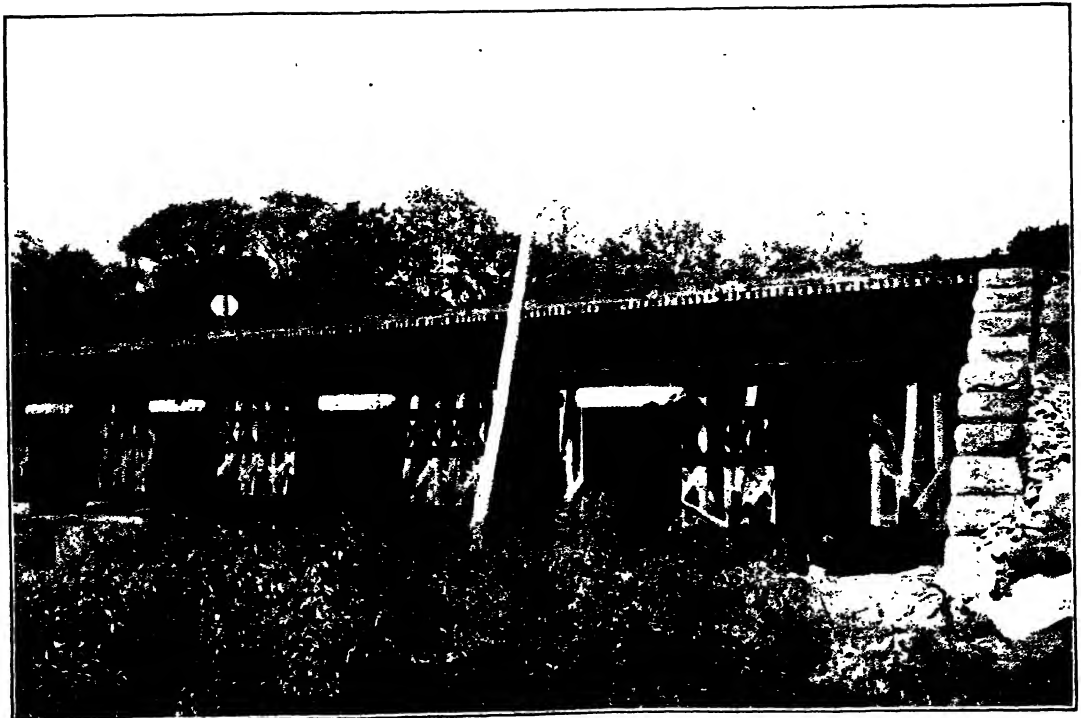


FIG. 142a.—East Channel Cylinder Piers of Chicago and Northwestern Railway Bridge at Clinton, Ia. 1908.

(Facing p. 454.)

CHAPTER XIII

CYLINDER AND PIVOT PIERS

ART. 142. GENERAL ARRANGEMENT

For light bridges the massive piers and foundations described in the preceding articles may furnish strength and stability far in excess of the requirements. This is due largely to the fact that the dimensions of the pier are governed not alone by the magnitude of the loads and the required bearing area on the foundations, but also by the distance between superstructure pedestals or base plates, size of pedestals and necessary edge distances. For this reason in many cases it may prove econom-

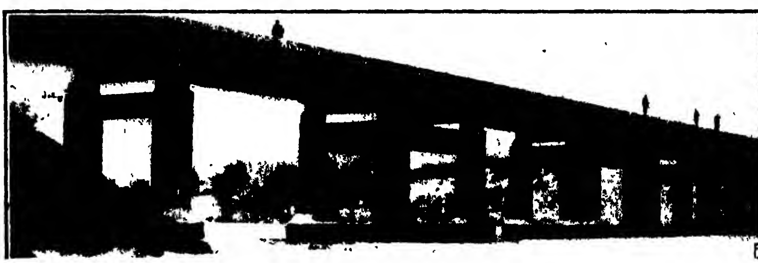


FIG. 142b.—Oxford Mill Pond Bridge, Chicago and Northwestern Railway.

ical to use cylinder piers. This type of pier consists of a number of long slender cylinders composed in most cases of steel shells filled with concrete. When used to support fixed spans, the pier consists of two or more cylinders in a line perpendicular, or nearly so, to the direction of the bridge, as illustrated in Fig. 142a; when used to support a viaduct, four cylinders are used, as illustrated in Fig. 142b; while for a pivot pier one cylinder at the center and a number of others on the circumference of a

circle are frequently used. Pivot piers are also formed of one large cylinder; this type is described in Art. 146.

The cylinders may be composed of concrete, brick or stone masonry. They usually have a metal shell of cast iron, wrought iron or steel. Wrought iron is not used at the present time.

Cylinders piers may be founded on bed rock or hardpan, on piles, open cylinder caissons, or pneumatic cylinder caissons. When founded on caissons, the pier is simply a continuation of the caisson, and as such is described in Arts. 88, 104 and 105. The following articles deal chiefly with the cylinder pier founded on bed rock or on piles, taking up only those features of the other two types which have not already been described.

ART. 143. METAL-SHELL CYLINDER PIERS

ON PILES.—Where the cylinders are of small diameter, piles are driven and the cylinder shells set over the same and filled with concrete. With the larger cylinders the shells are often placed before driving the piles. If the top stratum is composed of silt or other soft material, this should be excavated to a fairly solid material in order that the piles may have lateral support; care should also be taken to have the excavation carried below low-water level as well as to a depth free from any danger of scour. After excavating, the piles are driven and their tops cut off at some elevation above the surface of the ground. The cylinder shells rest on the river bottom or are sunk a few feet into the same. If clay is penetrated, it is sometimes possible to pump out the water and place the concrete filling in the dry; otherwise a few feet of concrete are placed in the bottom, and allowed to harden a few days; after which the cylinder is pumped out and the remainder of the filling placed in the dry. The concrete placed through the water should have about 20 percent more cement in it than that placed in the dry to allow for the washing-out action of the water.

The cylinder pier with pile foundations was first used in 1868 for the substructure of a bridge in Rhode Island, as stated in BAKER's Masonry Construction.

The Tensas River bridge in Alabama, built in 1870, was one of the early large structures using this type of foundation. The shells, of cast iron 1½ inches thick, had exterior diameters of 4 and 6 feet, and were in sections 10 feet long, the sections being united by bolts through interior flanges 2 inches thick and 3 inches wide. For the fixed spans of the bridge each pier was composed of two 6-foot-diameter cylinders 16 feet apart, while the pivot pier had a central cylinder 6 feet in diameter and six 4-foot cylinders arranged hexagonally on the circumference of a circle 25 feet in diameter.

Squared piles arranged closely together, with 12 in each of the 6-foot cylinders and five in the 4-foot ones, were driven to a depth of not less than 20 feet into the sandy bed of the river. Their tops were then tied together with bolts and sawed off at low-water level, 15 feet above the bed of the river. The cylinders were then sunk 10 feet into the bed of the river and enveloping the pile clusters, pumped out and filled with concrete.

The shells for the piers of the Victoria bridge in Nova Scotia, constructed in 1888, were made of wrought iron. The rim of the shells rested on piles cut off at the surface of the ground; other piles extended up into the cylinder as shown in Fig. 143*a*.[1] The piers were protected against scour and braced by cribs filled with stone and concrete, as well as by outside riprap.

Much larger cylinders than those above described were used in the Norfolk and Western Railroad bridge No. 5 across Elizabeth River at Norfolk, Va. The lower part of the cylinder for pier 2 consisted of a ⅜-inch steel shell 20 feet in diameter and 15 feet 9 inches long, stiffened by 3½- by 3½-inch angles spaced 5 feet apart vertically. A temporary upper section of the same diameter and high enough to reach to above water-level was attached to act as a cofferdam. The shell was then let down through the water, 23 feet deep at low tide, and sunk about 18 feet into the mud by dropping it a few times from a considerable height. The material was then excavated to the bottom edge of the shell after which 80 piles were driven, and

[1] Bridge Foundations in Nova Scotia, by MARTIN MURPHY, Transactions of the American Society of Civil Engineers, vol. 29, page 629, September, 1893.

cut off by a diver at an elevation 7 feet above the cylinder bottom. Concrete of a 1-2-4 mixture was then deposited through the water to within 6 inches of the tops of the piles.

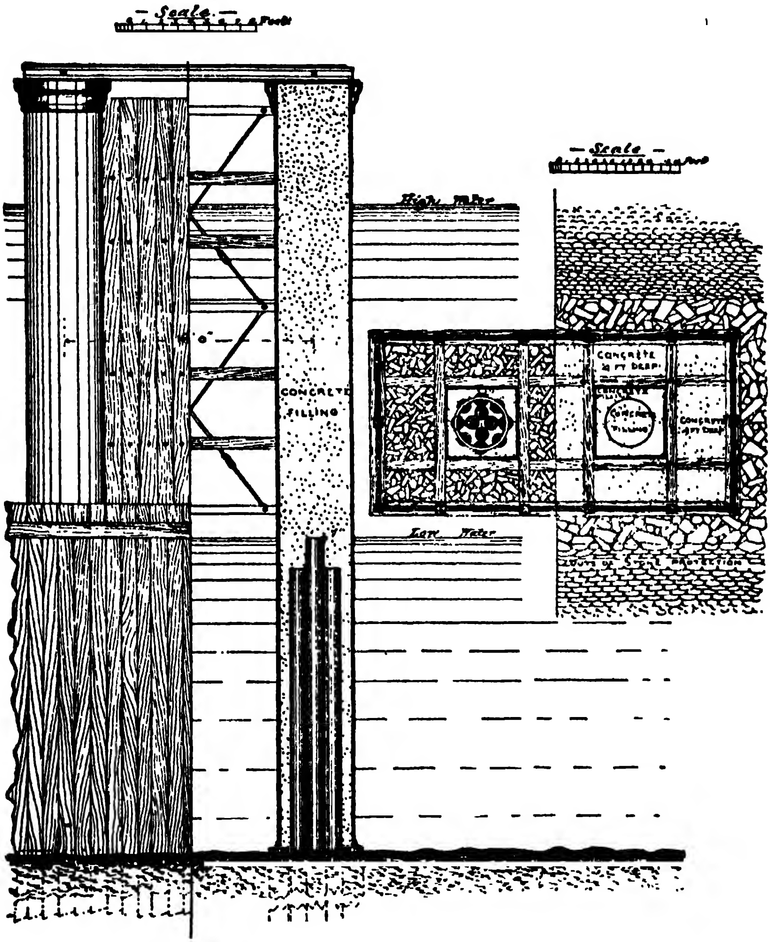


FIG. 143a.—Cylinder Piers of Victoria Bridge over Bear River, Nova Scotia.

After allowing this to set four or five days the cylinder was pumped out and a 2-foot layer of concrete, inclosing a grillage of rails, was placed over the tops of the piles to distribute the load more uniformly over them.

As shown in Fig. 143*b*, a cast-iron cylinder 10 feet in diameter was then placed in the larger cylinder. This shell was made in four lengths of 8 feet 9 inches each and each length was composed of four segments, the whole being bolted together through inside flanges. The metal was 1 inch thick. In the diagram the upper horizontal line represents the base of rail.

Round iron bars 1½ inches in diameter and 2½ feet long were run through the cast-iron shell near the bottom, and the outside cylinder was then filled with 1-2-4 concrete, which was crowned

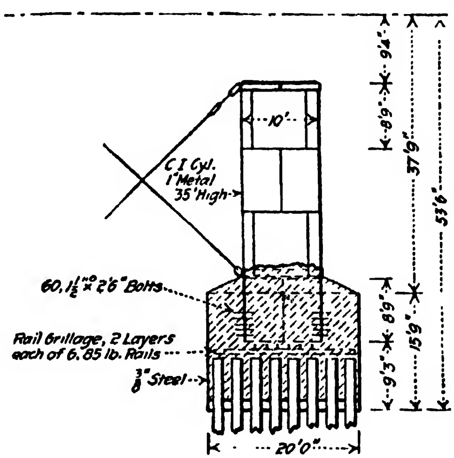


FIG. 143*b*.—Typical Cylinder Pier. Elizabeth River Bridge, Norfolk and Western Railroad.

up on a 30-degree slope. Concrete was placed in the 10-foot cylinder to within 2 feet of the top and a heavy beam grillage was set on this, crowned and grouted with concrete. The outside cofferdam was then removed. Cast iron was used for the upper part of the pier because of its better lasting qualities when only periodically immersed.

In the foregoing examples, in all cases some of the piles were extended well up into the cylinder. The advantage of this is the added stability against sliding and overturning. If the cylinders are not subjected to horizontal forces of any considerable magnitude, the piling may be cut at the base of the cylinder or lower. If this is done, the piles are surmounted with

a concrete capping or timber grillage and the cylinders placed on it.

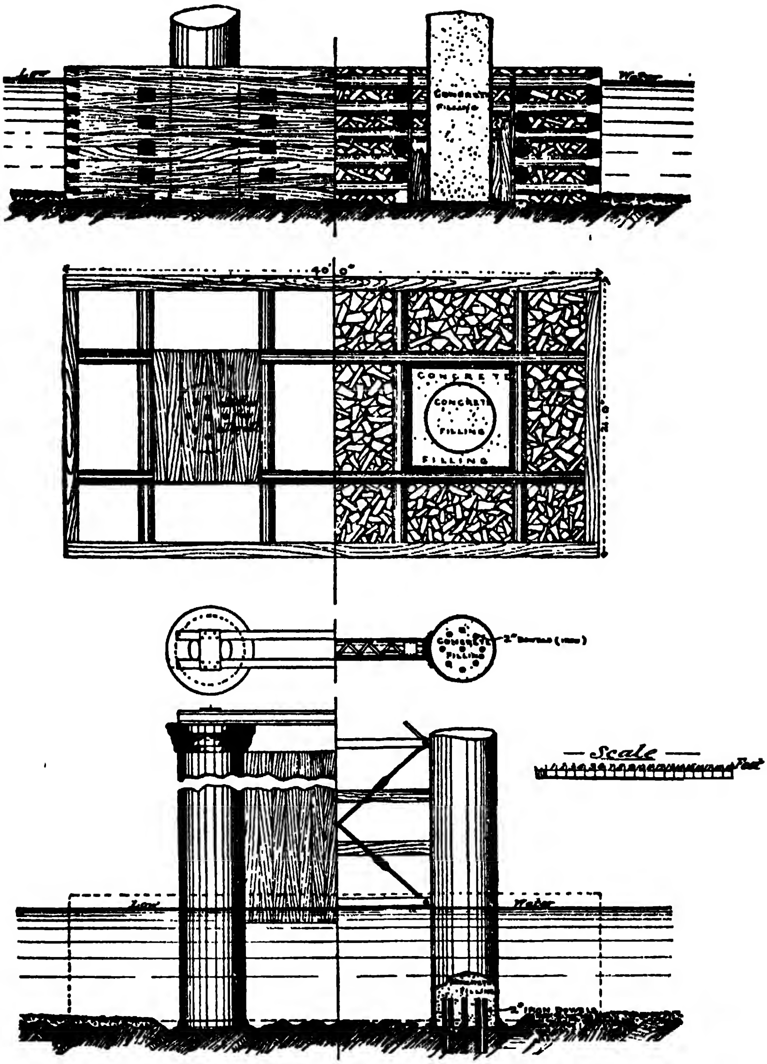


FIG. 143c.—Cylinder Piers of Avon River Bridge, Windsor, Nova Scotia.

The piers for the approach spans of the Cairo bridge of the Illinois Central Railroad were each formed of two 8-foot cyl-

inders. Since no water covered the site, a circular pit 8 feet deep was dug for each cylinder and 12 oak piles driven in it. The pits were then filled with concrete to the proper elevation, after which the cylinders were placed and concrete filled in around them to a depth of 6 inches. The cylinders were then filled with concrete.

ON ROCK OR HARDPAN.—Where the bottom is rock or hardpan, it is only necessary to clean and level off the site, place the cylinder and fill it with concrete. Where horizontal forces occur, the piers must be fastened to the foundation bed in some manner. This may be done by drilling holes in the latter and grouting rails or steel bars into the same, as was done for the piers of the Avon River bridge, Nova Scotia, illustrated in Fig. 193*c*.

ART. 144. DESIGN AND CONSTRUCTION

The size of the cylinder will depend on the load to be supported and the character of the foundation. The area of the base, with the pile foundation, is governed by the number of piles and their spacing, while if the pier rests on rock or hardpan the area is governed by the allowable bearing pressure on the same. The area of the upper part of the pier will depend upon the size of the pedestals or base plates of the bridge. In general, it is advantageous to have the diameter of the cylinder as small as possible, to avoid restricting the waterway and offering resistance to the current, ice and drift material. Where much ice and drift are present, it may be advisable to use a pointed nose, as illustrated in Figs. 144*a*, 88*b* and 89*b*.

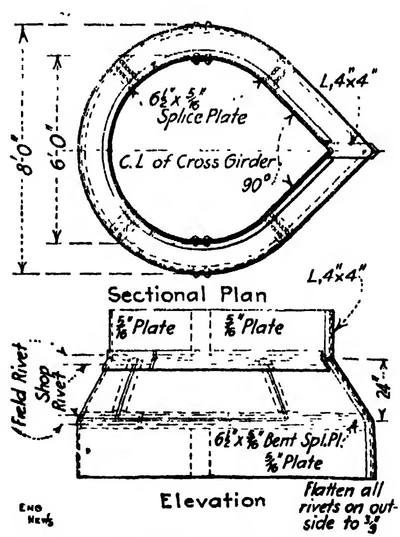


FIG. 144*a*.—Cylinder Pier with Pointed End.

Where the required diameters at the top and bottom differ materially, a shell having a smaller diameter at the top than at the bottom should be used. This may be done by using two separate shells, as indicated in Fig. 143*b*, or by a connection similar to Fig. 144*a*, or by using a shell in the form of a frustum of a cone, as illustrated in Fig. 104*a*.

The thickness of the shell is usually made just sufficient to take care of the stresses developed in handling and placing. Experience has demonstrated that it is inadvisable to use less then a 3/8-inch thickness, although a 5/16-inch thickness is often specified for highway bridge cylinders. A thickness of more than 1/2 inch will seldom be required even for large cylinders. Complete specifications and tables giving diameters and thickness of shells for highway bridge piers are given in COOPER'S General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges.

STABILITY OF PIERS.—The four possible methods of failure are undermining; settling, due to excessive pressure on the foundation; sliding; and overturning. Undermining is mentioned first because of the many failures of highway bridge piers in this country due to this cause. Where founded on caissons there is no danger from this source, but where founded on piles care should be taken to have the whole length of the piles well below any possible scouring action, otherwise the foundation may collapse through lack of lateral stability. If it is impracticable to get the piles down to such an elevation, cribs should be built around the tops as shown in Fig. 143*a*. The same protection should be given the foundation, where the material composing it is clay or hardpan or even the softer kinds of rock, if the piers are located in a scouring current.

To prevent settlement, the foundation, if composed of piles, should be designed in accordance with the rules given in Chap. III with regard to safe loads on piles; or if hardpan or rock, in accordance with safe unit-loads as given in Art. 184. The vertical load may usually be assumed as uniformly distributed over the base of the cylinder, for the transverse loads are resisted by a truss-like action of the cylinders and

bracing. Thus, with a two-cylinder pier, in addition to the vertical loads due to the live load, weight of superstructure and pier, there will be a downward vertical load on one cylinder equal to the moment of the transverse loads about the

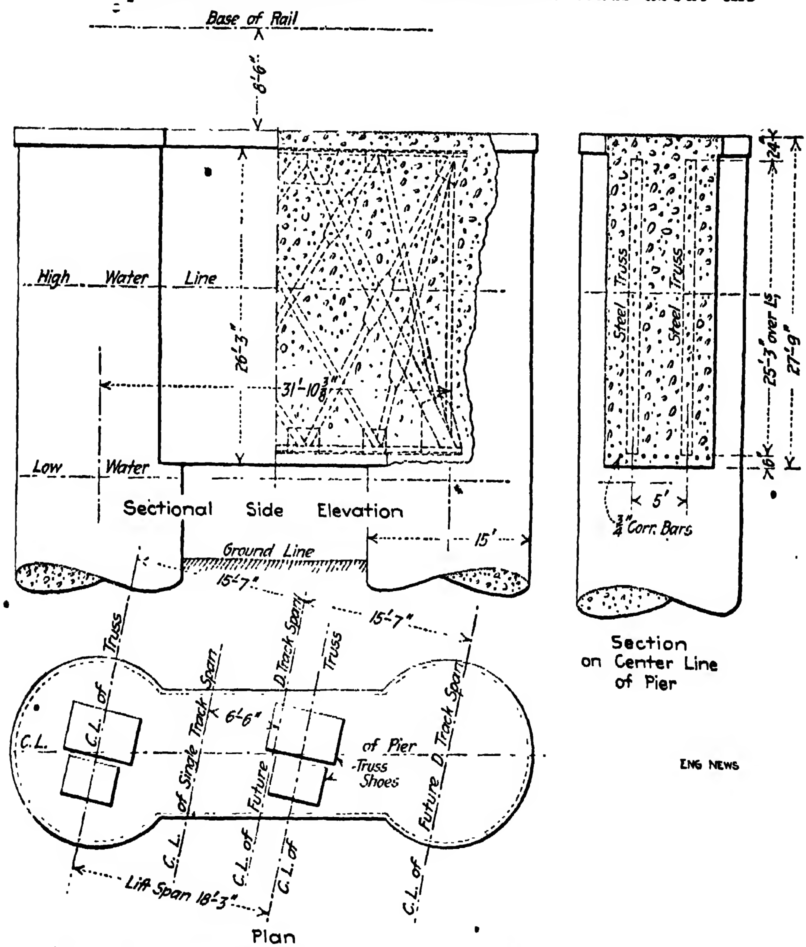


Fig. 144b.—Cylinder Piers Braced by a Truss Encased in Reinforced Concrete.

bottom of the pier divided by the distance between cylinders center to center.

Where forces exist tending to slide the pier, if a pile foundation is used, some of the piles should extend well up into the

cylinder; while if the cylinders rest on bed rock they should be anchored to the rock surface. A rock-filled crib placed around the cylinders, as illustrated in Figs. 143*a* and *c*, will add resistance to sliding.

To resist overturning, strong and rigid bracing should connect the cylinders. Many forms of bracing are illustrated in the accompanying figures; these include simple ties and struts of metal and wood, as in Figs. 143*a* and *c*; latticed girders, as in Fig. 142*a*; plate girders, as in Fig. 104*b*; double plate girders filled with concrete, as in Fig. 104*a*; and deep trusses embedded in concrete, as in Fig. 144*b*.

ART. 145. REINFORCED-CONCRETE CYLINDER PIERS

One of the disadvantages of the metal-shell type of pier is the necessity of keeping it painted. Although with steel shells it is not customary to design the shell to take any of the load, yet it is advisable to prevent the same from rusting for two reasons: First, the shell takes any tensile stresses that may develop in the pier due to eccentric loading, ice pressure, etc.; and, second, the appearance of the pier during the rusting of the shell, as well as the stained appearance of the concrete afterward, is unsightly.

The metal shell may be avoided by building the pier in forms in a cofferdam or by using a pre-molded shell of reinforced concrete. In either case the pier should be well reinforced with steel rods.

Each pier of a bridge at Buffalo, N. Y., on the Lake Shore and Michigan Southern Railroad was formed of two shafts, 30 feet 6 inches apart of centers, braced together with a steel girder encased in reinforced concrete. This girder served to transfer the load from the superstructure to the shafts. Each shaft, 13 feet 6 inches in diameter and about 51 feet high, rested on solid rock 36 feet below water-level. The shafts were constructed in cofferdams 18 feet in diameter, made of Lackawanna steel sheet-piling in 45-foot lengths. To within 21 feet of the top each shaft of the pier was octagonal in section and above this

circular. The reinforcement consisted of 80 vertical 1¼-inch corrugated bars extending from the top to a point 30 feet below and lying on the circumference of a circle 6 inches inside of the surface.

The strut, composed of a reinforced-concrete girder 4 feet 6 inches deep and about 5 feet wide, was reinforced with 48

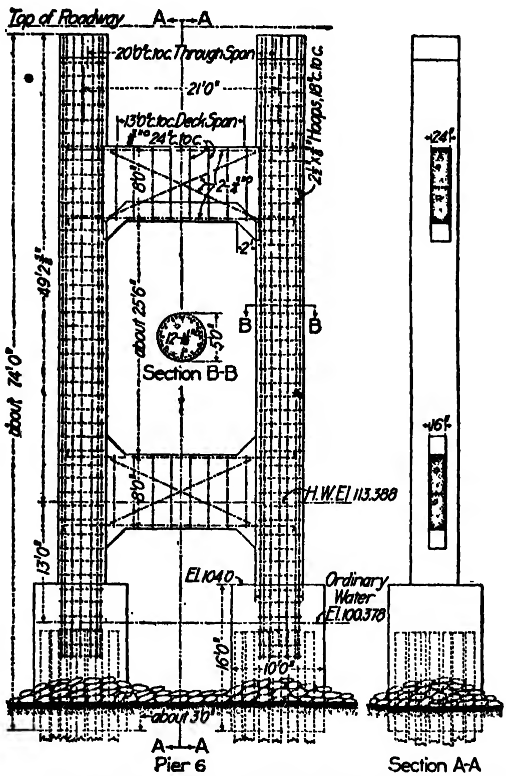


FIG. 145*a*.—Reinforced-Concrete Double-Cylinder Pier.

corrugated bars 1¼ inches square and 30 feet long, spaced about 6 inches center to center all around the strut, 6 inches from side and bottom faces and 12 inches below the top face. As stated above, the strut was further reinforced with a steel plate girder.

Figure 145*a* illustrates the reinforced-concrete cylinder piers of a highway bridge over the St. Croix River at Hudson, Wis.

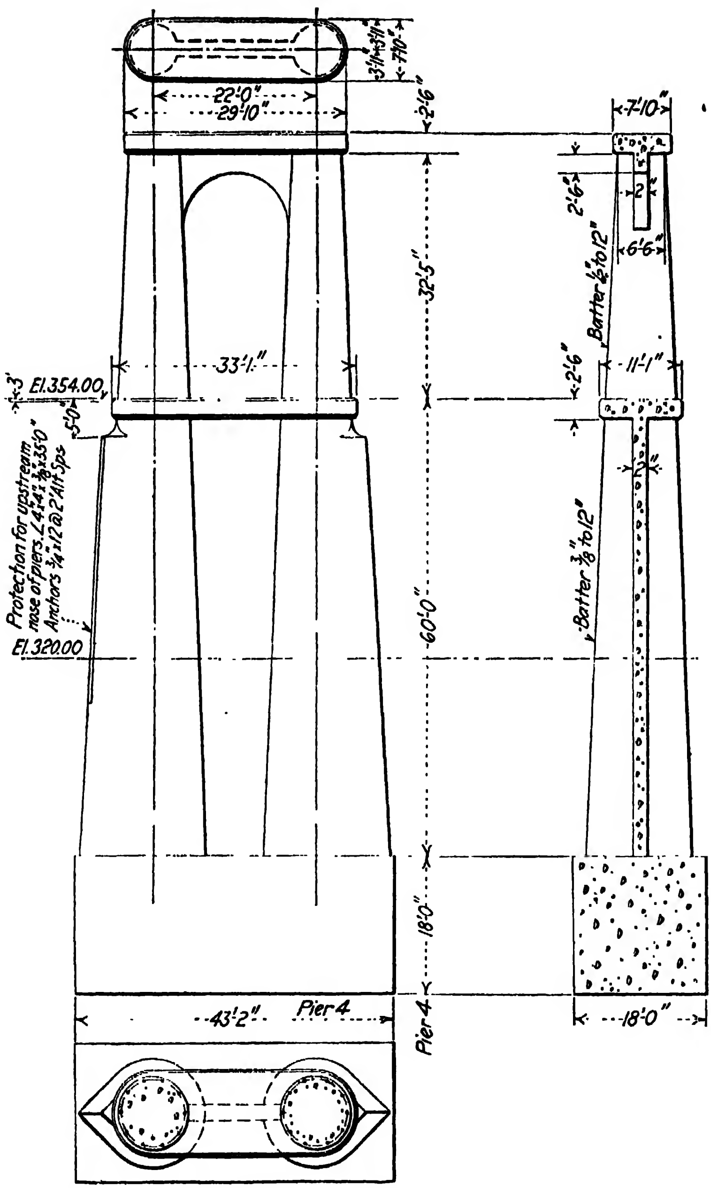


FIG. 145b.—Pier of the Kennewick-Pasco Bridge over the Columbia River.

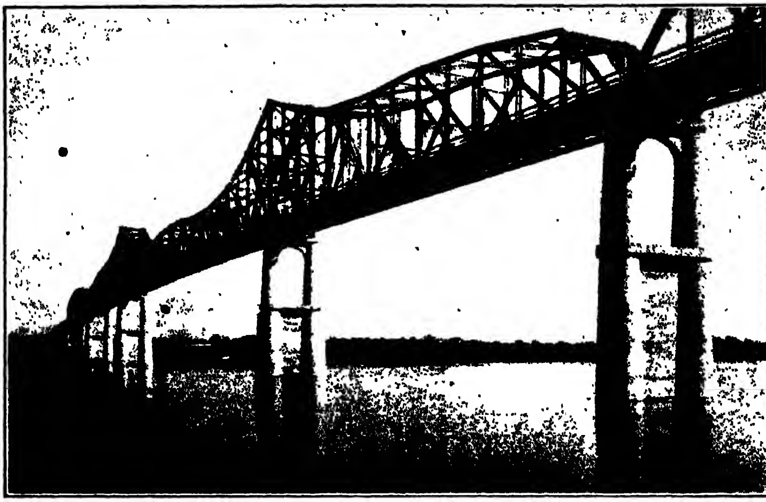


FIG. 145c.—Kennewick-Pasco Bridge over the Columbia River.

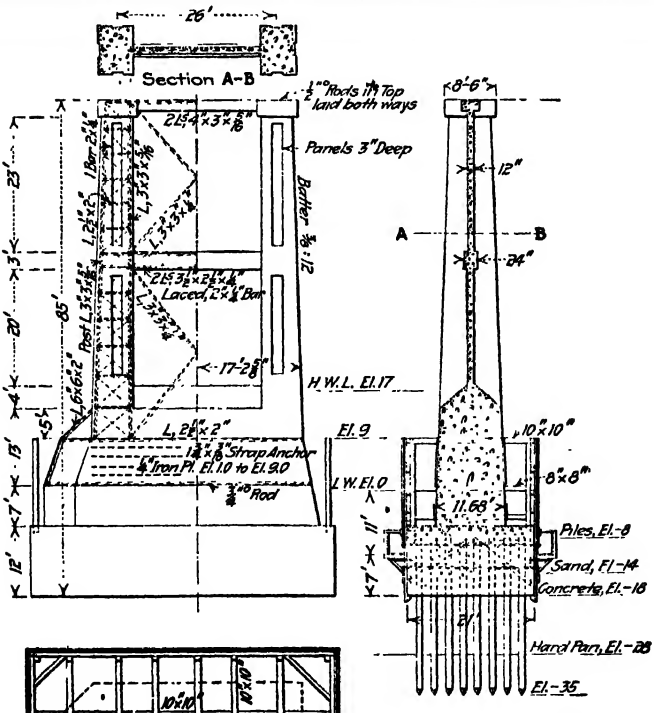


FIG. 145d.—Pier Reinforced with Structural Shapes.

Each pier consists of two reinforced-concrete shafts from 4 to 5 feet in diameter and braced together with reinforced-concrete webs. These shafts had separate pile foundations. The shafts were cast in wooden forms in 14-foot sections.

The piers used on the Kennewick-Pasco bridge over the Columbia River are of the dumb-bell type, as shown in Figs. 145*b* and *c*. The shafts are 6½ feet in diameter under the coping and have a batter of 1 in 24. Up to the elevation of high

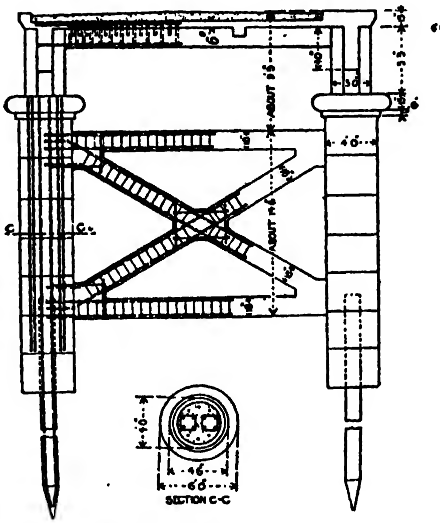


FIG. 145*e*.—Braced Bent of Reinforced-Concrete Cylinder Piers.

water the shafts are connected with a cross-wall of reinforced concrete 2 feet thick. Another cross-wall connects the shafts at the top. The coping at high water serves to break the appearance of extreme height. These shafts are reinforced with ¾-inch longitudinal rods and ½-inch hoops. In Art. 86 is shown another pier of the dumb-bell type.

In place of rod reinforcement, structural shapes are sometimes used for both shafts and diaphragms, as shown in Fig. 145*d*. Here, each pier consists of a pair of reinforced-concrete pedestals, paneled on three sides and connected with a 12-inch curtain wall or diaphragm. The structural steel was assembled and placed as a unit.

In constructing the piers for a bridge across the River Wanbeck, Stakeford, England, the cylinders were formed of sections of reinforced-concrete shells 48 inches in diameter placed over 14-inch pre-molded concrete piles driven into the river. The cylinders rested on the river bottom. After placing the shells, reinforcing rods were lowered into the same and the cylinders filled with concrete. The bracing was also of reinforced concrete. The structure is illustrated in Fig. 145*e*.

Art. 146. Large Cylinder or Pivot Piers

This type of pier, used almost exclusively for the center support of swing spans, resembles the cylinder pier in shape and the ordinary masonry pier in massiveness. The same types of foundations, kinds of material and methods of construction are used as for ordinary piers. Where protection against ice and drift is necessary, it is furnished by means of an independent pier, often constructed of long piles.

Figure 146*a* illustrates the all-concrete solid pivot pier on piles used for the St. Louis River bridge near New Duluth, Minn. The depth of water being about 28 feet, the pier was constructed in wooden forms inside of a circular steel sheet-pile cofferdam 36 feet in diameter. After driving the piles and cutting them off about 4 feet above the dredged bottom, a 6-foot layer of concrete was placed to form the 36-foot diameter footing course, the sheet-piling serving as a form for the sides. Above this footing course the form for the pier consisted of a wooden-stave water tank 16 feet high having a side batter of 1 inch to the foot (Fig. 146*b*). The second lift was made by raising the tank and planing a few of the staves to fit the new dimensions. For the coping course galvanized iron of the section shown in the illustration was used. A grillage of 24-inch I-beams distributed the load over the pier.

The pivot pier construction of the Dumbarton bridge of the Central California Railroad merits particular attention because of its simple solution of a difficult problem. At the site the depth of water at low tide was 51 feet and at high tide 58 feet,

with a maximum velocity of current of 4½ miles an hour. The bottom consisted of soft mud overlying stiffer material. On account of the great depth of water and velocity of current, the cofferdam process was not practicable and caisson foundations would have been expensive. Hence, it was decided to employ a metal shell with a pile foundation. The cylinder had a diameter

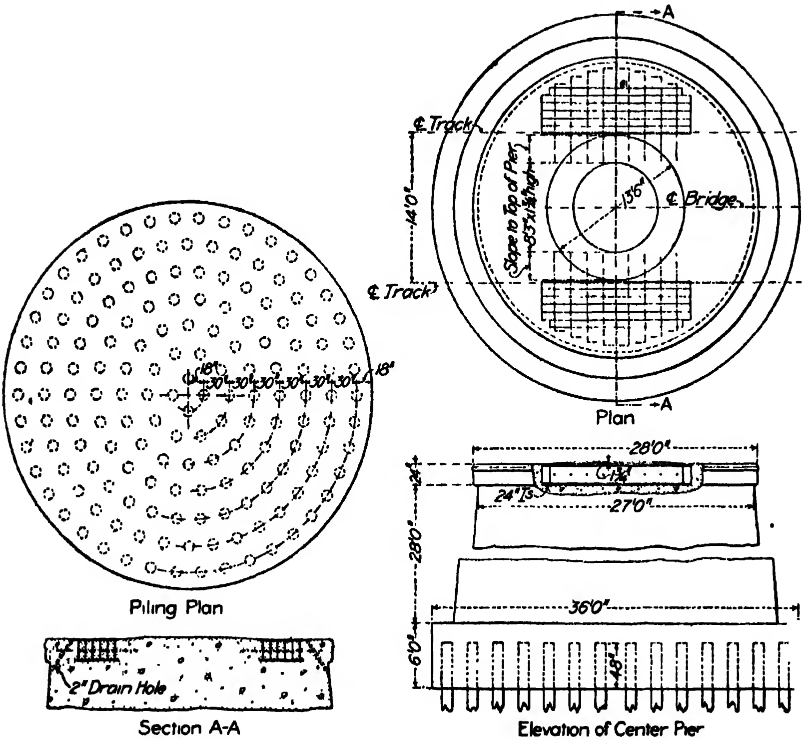
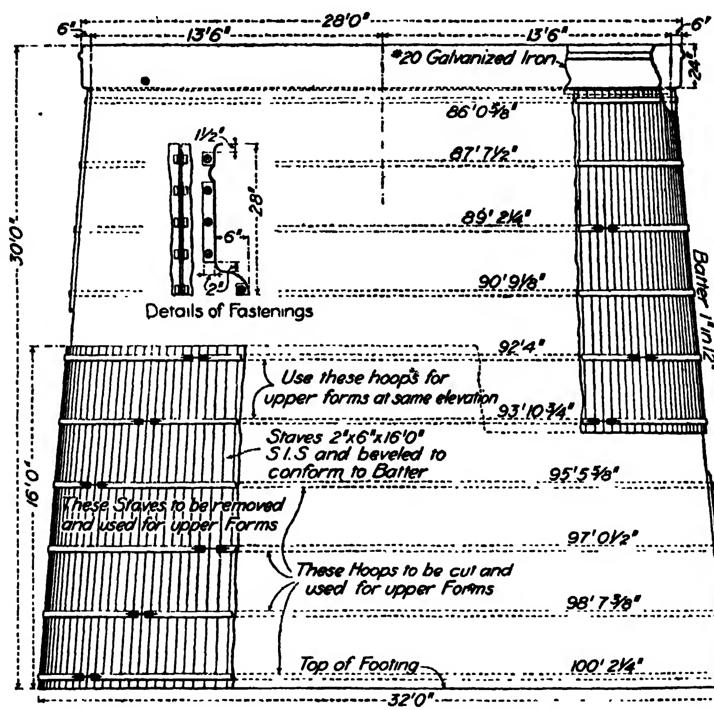


FIG. 146a.—Concrete Pivot Pier, St. Louis River Bridge, Duluth, Minn.

of 40 feet and was 72 feet 5 inches high in five vertical sections. After dredging out about 10 feet of the soft material on the bottom, the first section of the cylinder was lowered. This was effected by first lowering a guide frame of structural shapes and driving its feet into the bottom, after which the section of the cylinder was placed around this frame and lowered. Inside this section 141 piles were driven to a penetration of about 30 feet

and cut off below low-water level; some only about 3 feet below low water and others near the bottom. On completion of the pile driving, more sections of the shell were added, each section being filled with concrete placed through the water to within 7 feet of the top before another section was added. This was the highest level at which the top of the concrete would not be



Forms for Concrete Center Pier

FIG. 146*b*.—Form for Constructing Concrete Pivot Pier.

disturbed by the tidal current passing over the top. Further details of this interesting work may be found in an article by E. J. SCHNEIDER in Transactions of the American Society of Civil Engineers, vol. 76, page 1572, December, 1913, entitled "Construction Problems, Dumbarton Bridge, Central California Railway;" or in Engineering Record, vol. 62, page 172, Aug. 13, 1910.

Where the lateral forces on the piers are small, it is not necessary to extend the piles into the cylinders. In the construction of a pivot pier in the Willamette River, Portland,

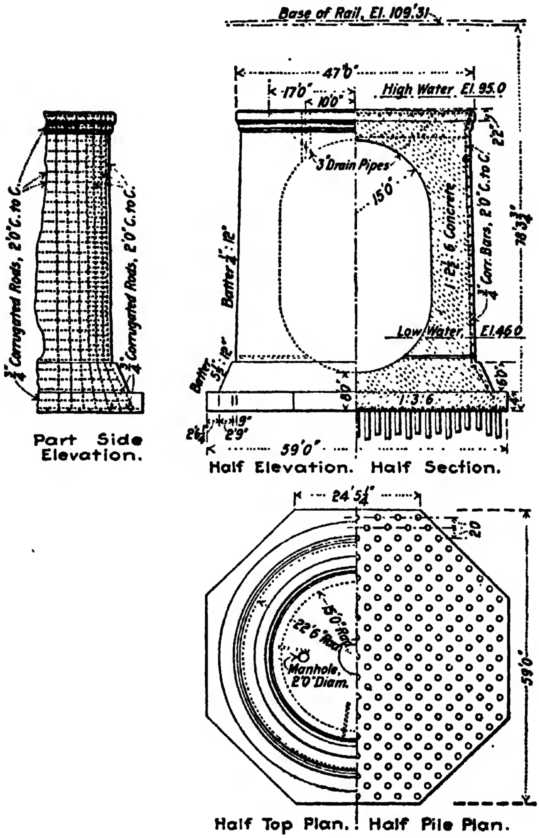


FIG. 146c.—Reinforced-Concrete Pivot Pier, Illinois Central Railroad Bridge. Gilbertsville, Ky.

Ore., where the depth of water was 60 feet, piles were driven and cut off near the bottom. A timber grillage extending to within 3 feet of low-water mark was placed on the piles. On

this grillage was placed a steel shell 46 feet in diameter about 30 feet high, which was filled with concrete to form the pivot pier.

Figure 86*c* shows the pivot pier of the Chelsea Bridge North, Boston, which was faced with stone masonry. The foundation for this pier is described in Art. 86.

As in ordinary bridge piers, the tendency at present is to make the pivot pier of hollow construction, leaving out masonry from that part of the pier that is but slightly stressed.

Figure 146*c* shows the reinforced-concrete hollow pivot pier of the Tennessee River bridge of the Illinois Central Railroad. The hollow space is domed at the top and bottom. The entire load from the superstructure comes on the pier through a cast-iron track 38 feet in diameter. The circular center line of the 8-foot wall has the same diameter, thus avoiding eccentric stresses in the pier.

CHAPTER XIV

BRIDGE ABUTMENTS

ART. 147. FORM AND DIMENSIONS

A bridge abutment is a masonry structure at one end of a bridge used for the double purpose of transferring the loads from the bridge superstructure to the foundation and to give such lateral support to the adjacent embankment as is required to maintain it in position.

The abutment serves both as a pier and as a retaining wall. Because of the latter function, involving as it does the question of earth pressure, a mathematical treatment on the design of abutments has not yet been developed in a satisfactory manner.

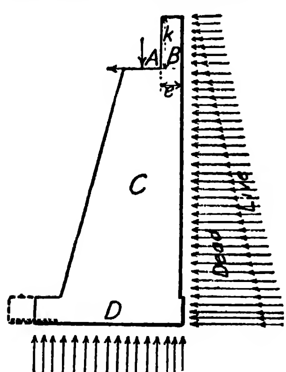


FIG. 147*a*.—Section of Abutment Indicating the External Forces Acting upon It.

Figure 147*a* shows a typical section of an abutment through its center and parallel with the direction of the bridge. *A* is the bridge seat, which consists of a horizontal surface on which rest the end bearings of the superstructure; *B* is the back or parapet wall, which supports the upper part of the embankment and prevents the same from spilling down on the bridge seat; *C* is the main body or stem of the abutment; and *D* is the footing.

The forces acting upon the abutment are as shown in the illustration: first, the loads on the bridge seat, which consist of a vertical force from the live and dead loads from the superstructure and a horizontal force due to traction and in some cases to expansion and contraction of the bridge spans; second,

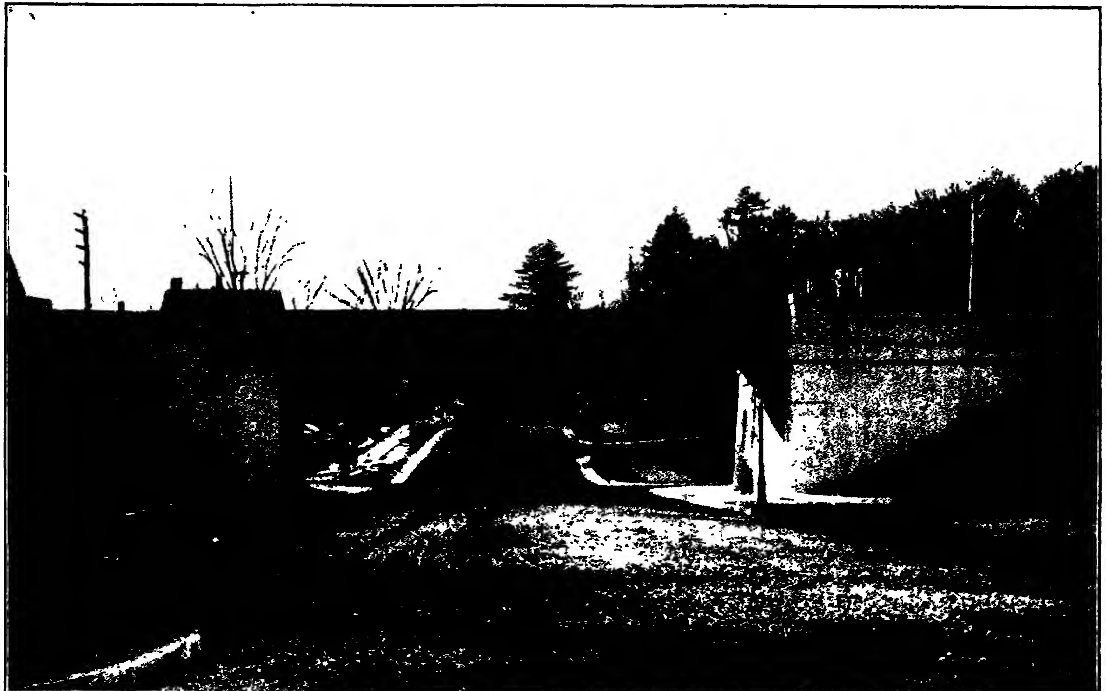


FIG. 147*b*.—Boston and Maine Railroad Bridge over Lynn Fells Parkway at Melrose, Mass., Showing the Aesthetic Effect of U-Abutments Designed by the Metropolitan Park Commission. The Bridge was Completed in 1907.

(*Facing p.* 474.)

FIG. 148a.—Effect of Unequal Bearing on Foundation Bed of Abutment.

the earth pressure from the embankment against the back of the wall, due both to the weight of embankment and weight of live load; third, the weight of the abutment; and, fourth, the reactions from the foundation.

Abutments are classified according to their form. The three original types are the wing-wall abutment, the U-abutment and the T-abutment. At present there are many modifications of these fundamental types. In the wing-wall abutment the wings, which may be parallel with the face of the wall or at any angle with the same, serve merely to keep the embankment material from slipping into the stream or moving out into the roadway, as shown in Figs. 149*a* and *b*. In the U-abutment the wings are made parallel with the roadway. The front wall is usually located at a point such that the side embankment material, having a slope of 1½ on 1 or 1 on 1, will not extend out beyond the face of the abutment (Fig. 147*b*). The T-abutment has the same form of bridge seat as the wing-wall and U-abutment, but instead of wings it has a solid stem which supports the track or roadway back to a point at which the embankment is sufficiently high to support it. This type is illustrated in Figs. 150*f* and *g*.

Abutments may be of the square or skew type, depending on the angle the face wall makes with the axis of the roadway. Figure 150*f* represents the skew type, the other illustrations being of the square type.

DIMENSIONS OF ABUTMENTS.—Like the bridge pier, the dimensions of the bridge abutment will vary with many conditions, such as class of superstructure, height of abutment, type of foundation and kind of embankment. However, owing to the uncertainties involved in the design, certain dimensions are being standardized to a large extent.

The dimensions of the top of an abutment will depend on the same factors as the top of a bridge pier, except that, where in the latter case bearing must be furnished for two trusses or girders, in the former case there must be a width sufficient for one truss or girder bearing plus a distance *e* (Fig. 147*a*) which is necessary to furnish the required stability to the parapet wall. The

TABLE No. 147a[1]

Span	Thickness of abutment under coping = thickness of back wall + figures given below					
	Class A		Class B		Class C	
	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.
25	2– 0	2– 2	2– 0	2– 0	2– 0	2– 0
50	2– 2	2– 9	2– 0	2– 2	2– 0	2– 0
75	2– 6	3– 3	2– 0	2– 6	2– 0	2– 0
100	2– 8	3– 6	2– 0	2– 8	2– 0	2– 2
125	2–10	3– 9	2– 2	2–10	2– 0	2– 4
150	3– 0	4– 0	2– 4	3– 0	2– 0	2– 6
175	3– 2	4– 3	2– 6	3– 2	2– 0	2– 8
200	3– 4	4– 6	2– 8	3– 4	2– 2	2–10
250	3– 8	5– 0	2–11	3– 8	2– 5	3– 2
300	4– 0	5– 6	3– 1	4– 0	2– 7	3– 6
350	4– 4	5–10	3– 3	4– 4	2– 9	3–10
400	4– 8	6– 2	3– 5	4– 8	2–11	4– 0

Span	Length of abutment under coping = distance center to center of trusses + figures below					
	Class A		Class B		Class C	
	S. T.	D. T.	S. T.	D. T.	S. T.	D. T.
50	3–6	4–0	3–6	3–6	3–6	3–6
100	4–0	5–0	3–6	4–0	3–6	3–6
150	4–6	5–6	4–0	4–6	3–6	4–0
200	5–0	6–0	4–0	5–0	3–6	4–6
250	5–0	6–6	4–6	5–0	4–0	4–6
300	5–6	7–0	4–6	5–6	4–0	5–0
350	6–0	7–6	4–6	6–0	4–6	5–0
400	6–0	7–6	5–0	6–0	4–6	5–6

Note: All dimensions are expressed in feet and inches. S. T. = single track; D. T. = double track. Class A, heavy traffic; Class B, medium traffic; Class C, light traffic.

[1] From article by C. C. SCHNEIDER in STREET RAILWAYS JOURNAL, Sept. 15, 1906.

value of e is usually taken as 0.40 or $0.45k$ unless the parapet wall is reinforced.

[1]"The width of bridge seats, exclusive of the projection of the coping, shall be at least 12 inches greater than required for the bed plates of steel superstructures, and the length of bridge seats shall not be less than the total width of bridge out to out of bearings plus 4 feet. The upper surfaces of the back and slope walls shall not have a less width than 2 feet for railway and 1½ feet for other bridges. The thickness of coping shall not be less than 18 inches for railway, and 12 inches for other bridges."

Table No. 147*a* gives the approximate minimum dimensions of thickness and length under coping for electric-railway bridges.

The thickness of the stem may be designed in accordance with the methods outlined in Art. 148. However, owing to the uncertainties involved in estimating the earth pressure, as well as to the possible large forces resulting from the freezing of water in the embankment, the thickness at any point should not be made less than 0.4 the height at that point. Some experienced engineers specify a coefficient of 0.5 where the abutment rests directly on soil. GREINER states that[1] "the thickness of the stem or back wall at any elevation shall not be less than 0.45 of the height of the masonry above that elevation for steam railway bridges, and 0.4 for other bridges."

ART. 148. DESIGN AND CONSTRUCTION

Abutments may be built of stone masonry, concrete or reinforced concrete. For the reasons given in Art. 137, stone masonry is but little used at present. A facing of stone is sometimes used, but not to the extent that it is used for piers, since abutments are usually not subjected to the action of the current, with its accompanying ice and drift material. Where built of concrete it is advisable to use a small amount of surface reinforcement for the case reasons as those given for piers in

[1] General Specifications for Bridges, Part III, by J. E. GREINER.

Art. 137. According to GREINER, "The surface bonding reinforcement shall be the same as provided for piers, but no horizontal layers of network will be required."

Solid massive abutments may be made with 1-2½-5 to 1-3-6 concrete below the coping, with a 1-2-4 mixture for copings and parts above the same. For reinforced-concrete abutments all concrete should be a 1-2-4 mixture; or, better, one part of cement to six parts of aggregate (before combining the sand and stone), the sand and stone being in such proportions as will give the densest concrete as determined by trial mixtures.

Proper drainage behind the abutment is of primary importance, since earth in a semi-fluid condition exerts a pressure much greater than ordinary earth pressure. Water collecting behind the abutment and freezing is almost sure to result in a cracking of the abutment. Weep holes through the walls near the ground line, connecting to French drains along the back, make a good drainage system.

The filling back of the abutment should be placed in horizontal layers about 1 foot thick and thoroughly tamped or rolled before the next layer is placed. This will avoid the wedging action which results if earth is dumped from a height.

DESIGN OF ABUTMENTS.—The vertical loads to be sustained on any horizontal plane are the live load, impact load, weight of superstructure and weight of abutment above the plane in question. Impact may usually be neglected.

The lateral forces parallel to the axis of the bridge are the tractive force and the pressure from the embankment due to both the weight of the embankment material and the live load. At right angles to the axis of the bridge are the wind loads from the superstructure and on the abutment. The latter two are usually neglected, their effect being slight compared with the other forces.

Of all the forces coming on the abutment the earth pressure from the embankment is the most uncertain in its effect and the most difficult to analyze. For descriptions of various methods of computing earth pressure, the reader is referred to HOWE's Retaining Walls for Earth; CHURCH's Mechanics of

Engineering; and Turneaure and Maurer's Principles of Reinforced-Concrete Construction. The other forces, with the exception of the weight of the pier, will be the same as those used in designing the superstructure.

For stability, the solid-gravity abutment must satisfy the same conditions as those given for piers in Art. 140. For reinforced-concrete abutments the base must satisfy these same conditions while the constituent parts of abutments are designed as beams and columns.

Unless the abutment rests on rock or some other unyielding material, it is not entirely satisfactory to have the resultant cut the base just within the middle third; it should be close to the center in order to give an approximately uniform pressure over the whole base, because a slight unequal settlement causes a considerable lateral movement at the top, giving a condition illustrated in Fig. 148*a*. If the back face of the abutment is vertical and at the same time the dimension *e* (Fig. 147*a*) is diminished by reinforcing the parapet wall with vertical rods near the rear surface, and the footing is extended and reinforced near the bottom as shown by the dotted lines in Fig. 147*a*, the pressure may be made to strike the base near the center.

Figure 148*b* shows the design of abutment 5 of the Beaver bridge of the Pittsburgh and Lake Erie Railroad. The unit-pressures due to the different resultants are given in the following table:

Table 148*a*. Unit-Pressures (Fig. 148*b*)

No.	Loading	Max.	Min.	Mean
1	Masonry unloaded, no fills.................	2.3	0.9	1.6
2	Masonry and fill at back......	2.3	1.5	1.9
3	Masonry, dead load and fill at back.......	2.5	2.1	2.3
4	Masonry, dead load, live load and fill at back	2.7	2.5	2.6
5	Masonry, dead load, live load and fill at back and front.	3.2	3.0	3.1
6	Masonry, dead load, live load, fills back and front, earth inside.	3.6	3.2	3.4

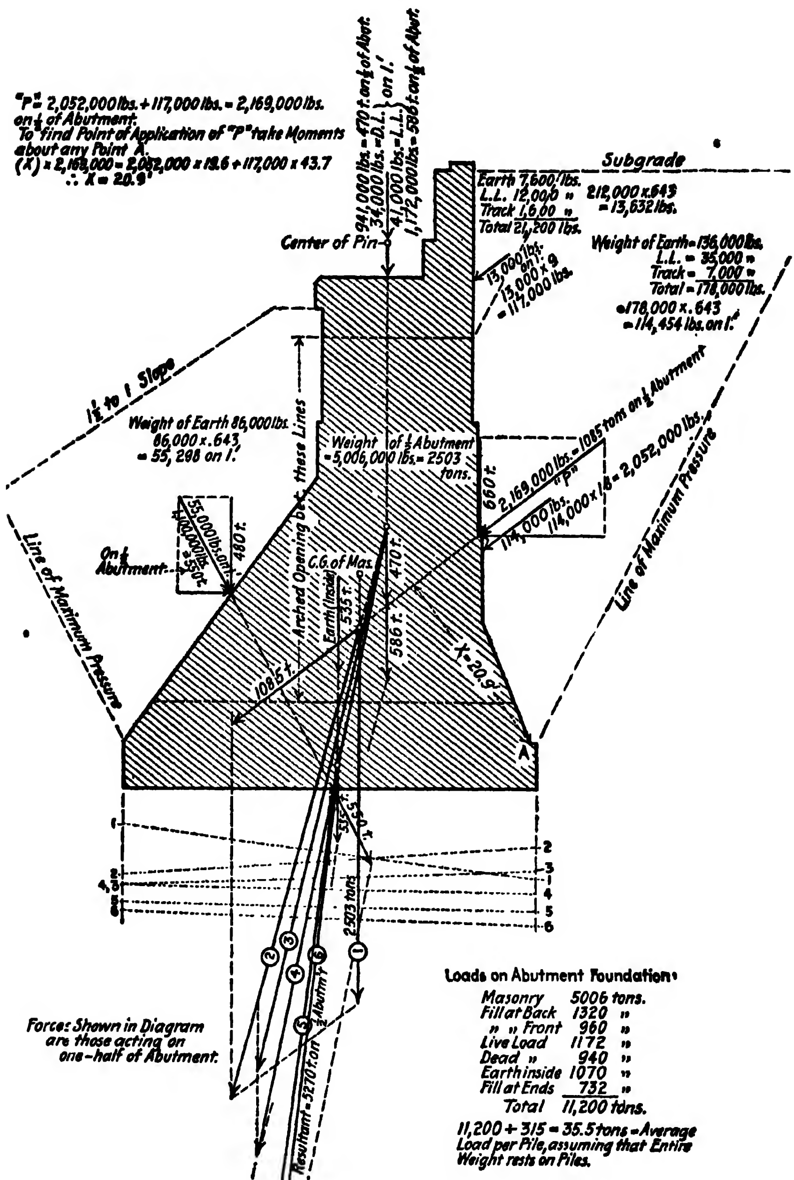


FIG. 148b.—Diagram of Forces Acting on North Abutment and Its Foundation. Pittsburgh and Lake Erie Railroad Bridge over Ohio River at Beaver, Pa., 1908–10.

ART. 149. WING-WALL ABUTMENTS

This type of abutment usually has its wings parallel to, and in the line of, the face or front wall of the abutment when used for street crossings, as shown in Fig. 149*a*, while for river crossings the wings are usually at an angle with the front face. The advantage of deflecting the wings in the latter case is that the abutment is thus better protected from water getting in behind the same, and it also allows the current to pass with less disturbance. It is not customary to extend the wings to the toe of the embankment, they being stopped some distance back of this point and the material allowed to spill out in front of the ends. Where the stream is liable to scour away this material, it should be riprapped as shown in Fig. 149*b*.

The length of wings will be a minimum when the face of the wing bisects the angle between the face of the front wall and the shoulder of the fill. For square abutments, according to BAKER,[1] the proper angle of deflection for the wings, for a minimum quantity of masonry, is between 25 and 35 degrees from a line through the front face of the abutment, if the earth flowing around the toe is to be kept 3 or 4 feet back of this line. An angle of 30 degrees is widely used.

The wings are designed as retaining walls, but, due to the uncertainties of the earth pressure theories, many engineers specify a relation of base thickness to height of wall. This ratio ranges from 0.33 to 0.45, a value of 0.4 being widely used.

Figure 149*c* illustrates a typical wing-wall abutment. The exposed faces have steel reinforcement as a protection against cracking through expansion and contraction of the concrete near the outside. Reinforcement is also placed just above the pile foundation to distribute the load more uniformly over the same. The design of this abutment would have been improved if the bridge seat had been moved a short distance to the right by narrowing and reinforcing the parapet wall, and the footing had been moved a short distance to the left. The only added

[1] Masonry Construction, 10th edition, page 536.

expense in so doing would have been in the slightly increased cost of the superstructure.

Figure 149*d* illustrates the type of wing-wall abutment used by the Baltimore and Ohio Railroad for both straight and splayed wings. Table No. 149*a* gives the yardage of concrete for this type of abutment with straight wings, these figures being based on a depth of footing of 6 feet, a bridge seat width of 3 feet and for two widths of fill, 18½ and 31½ feet.

TABLE 149*a*.[1] YARDAGE IN RAILROAD BRIDGE ABUTMENT. WINGS PARALLEL TO FACE OF ABUTMENT

	Height *H*	10	14	18	22	26	30	34	38	42
Width of fill, 18½ ft.	*B* = 4 ft.	101	171	270	400	561	757	984	1258	1569
		8.7	13.8	20.6	28.2	37 9	49.3	66.0	72.0	84.3
	B = 6 ft.	....	170	271	399	557	747	985	1258	1570
			14.2	21.1	28.8	37.5	47.3	60.2	72.3	84.8
	B = 8 ft.	.....	182	279	410	575	768	1008	1275	1599
			15.0	21.4	28.8	38 3	48.0	61.0	71.3	85.4
Width of fill, 31½ ft.	*B* = 4 ft.	142	231	354	511	694	932	1199	1512	1855
		12.0	18.2	26.0	34.7	45.7	58.0	69.7	82.6	96.2
	B = 6 ft.	.....	226	349	505	694	918	1189	1505	1860
			18.6	26.6	35.3	45.3	56.0	70.2	83.0	96.7
	B = 8 ft.	.....	240	357	517	713	938	1216	1522	1890
			19.3	26.9	35.3	46.1	56.6	70.9	82.1	97.3

[1] Compiled from tables in Foundations, Abutments and Footings, by HOOL AND KINNE, New York.

For each value of *B* the upper row gives the quantity of concrete in cubic yards for one abutment with a depth of footing of 6 feet, while the lower row gives the yardage for each additional foot of depth.

Figure 149*e* shows the standard design of wing-wall abutment for steel bridges of the Ontario Department of Public Highways. The dimensions of ends of wing walls are the same for all heights of abutments, while the lengths of wing walls as well as value to be added to top widths of face wall to get dimensions of *A* and *B* are given below. The yardage of abutment is also given, these values being for a 16-foot roadway.

FIG. 149a.—New York Central Railroad Belt Line Bridge over Colvin Street, Buffalo, N. Y., 1910.

(Facing p. 482.)

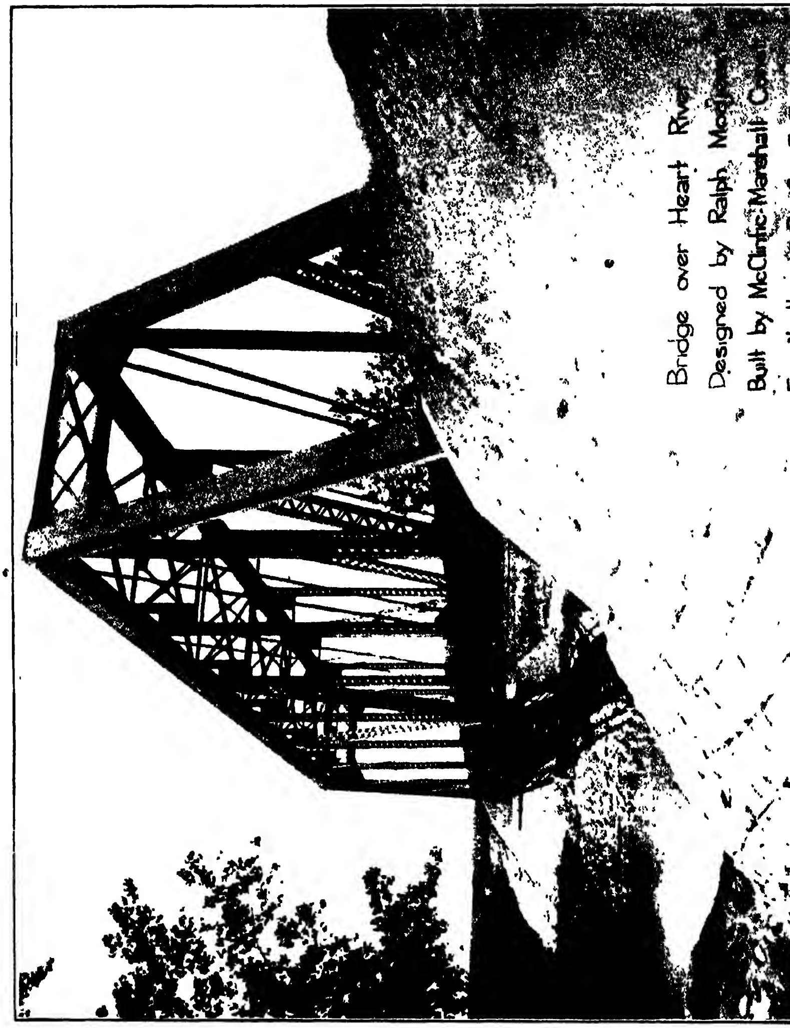


FIG. 149*b*.—Northern Pacific Railway Bridge over Heart River at the Fourth Crossing, 4 Miles West of Mandan, North Dakota, Showing Abutments and Paved Protection of Embankments. Completed in 1905.

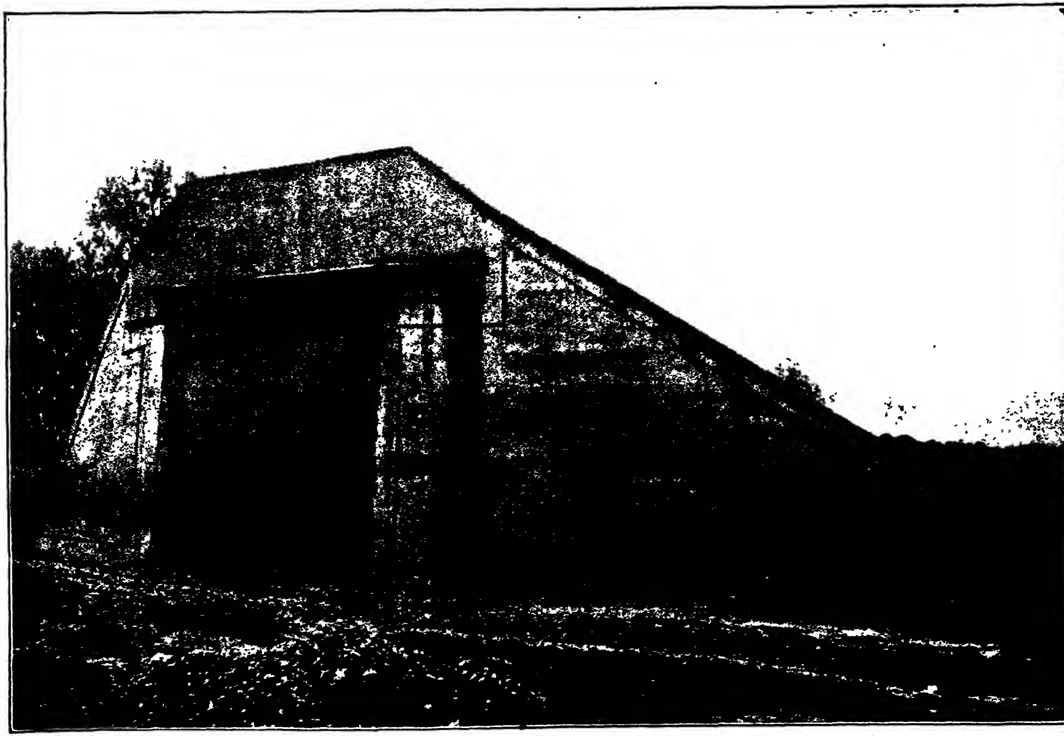


FIG. 149*f*.—Reinforced-Concrete Abutment, Wabash Railroad Viaduct, Monticello, Ill., 1903.

FIG. 149g.—Rear View of Reinforced-Concrete Abutment at Monticello, Showing Counterforts.

Height H, feet	10	12	14	16	18	20	22	24	26	28
Length wind wall, feet and inches....	6–0	9–6	13–4	16–8	20–0	23–9	27–6	30–6	34–2	37–8
Add to top width to get A, inches....	1	2	3	4	5	6	7	8	9	11
Add to top width, to get B, inches...	6	7	8	9	10	11	12	13	14	16
Concrete, cubic yards..............	42	61	83	107	140	184	232	287	357	430

For an 18-foot roadway the quantities should be increased from 3 cubic yards for a 10-foot height to 15 cubic yards for a 28-foot height.

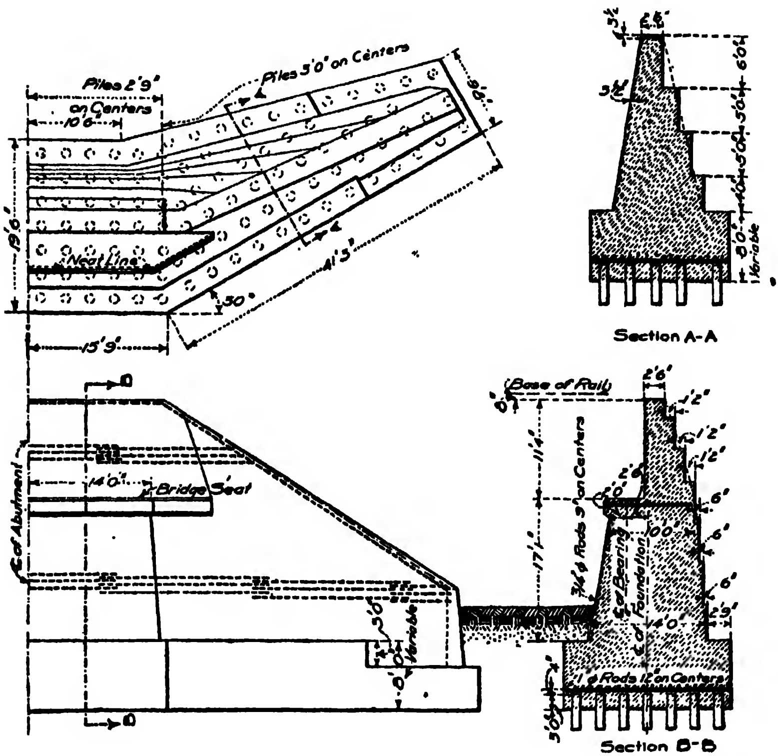
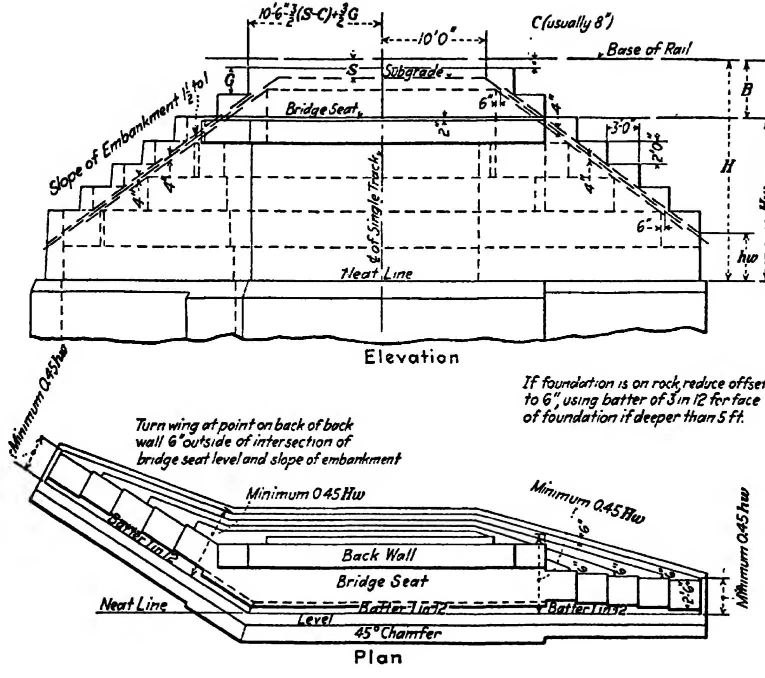


FIG. 149c.—Abutment of Peoria and Pekin Union Railway over Illinois River at Peoria, Ill.

For all heights the thickness of the stem at the neat line is made not less than four-tenths the height of abutment above



Values of V and T

H or Hw	V	T
6' - 10.9'	6"	1'-6"
11' - 15.9'	1'-0"	2'-0"
16' - 20.9'	1'-6"	2'-6"
21' - 25.9'	2'-0"	3'-0"
26' - 30.9'	3'-0"	4'-0"
31' - 40.9'	3'-0"	4'-6"
41' - 50.0'	3'-0"	5'-0"

Values of D and P

UC to NL	D	P
Under 25'	2'-2"	4"
25' - 34.9'	2'-4"	4"
35' - 49.9'	2'-6"	6"
50' and over	[illegible]	6"

FIG. 149*d*.—Standard Wing-Wall Abutment, Baltimore and Ohio Railroad.

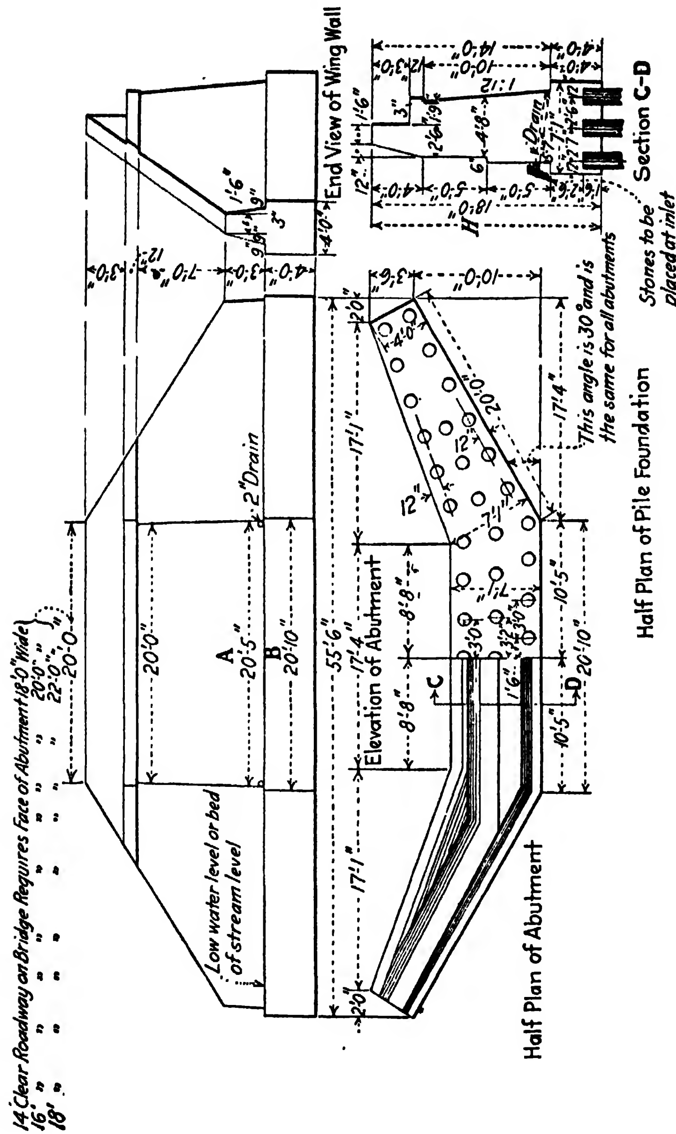


FIG. 149e.—Standard Abutment, Ontario Department of Public Highways.

neat line, this thickness being obtained, where necessary, by a series of steps along the back face, the number of steps varying from 1 where H is 18 feet to 5 where H is 28 feet. The width of bridge seat and the height of back wall vary with the type and length of span.

For high abutments the reinforced-concrete buttressed abutment will show some economy over the solid type. The first structure of this type was designed by A. O. CUNNINGHAM, in 1903, for a bridge on the Wabash Railroad at Monticello, Ill. Figures 149*f* and 149*g* show views of this abutment from the front and rear, respectively, while Fig. 149*h* shows a section of the same. As is seen in these illustrations, the abutment consists of a floor, face wall, bridge seat, parapet wall and buttresses. For stability this type requires a wider base than the solid section abutment, for here earth filling instead of concrete contributes a large part of the stability.

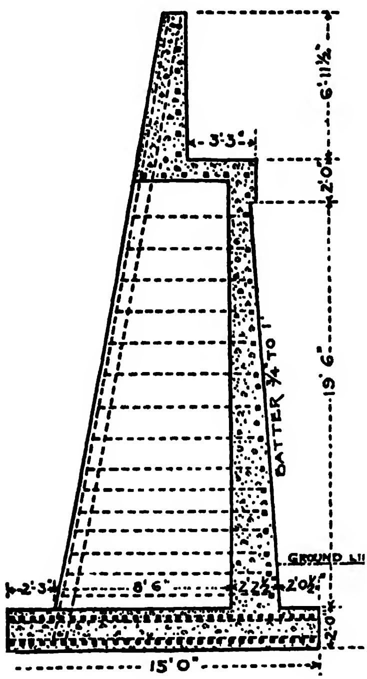


FIG. 149*h*.—Section of Reinforced-Concrete Abutment.

The design of the buttressed abutment is similar to that of the buttressed retaining wall, the main difference being that in the former the buttresses under the bridge seat serve to carry, as columns, the weight of the superstructure, as well as acting as beams to resist the earth pressure. For the outline of, as well as an example of, the design of buttressed and cantilever retaining walls, see TAYLOR and THOMPSON'S Concrete, Plain and Reinforced.

ART. 150. U-ABUTMENTS AND T-ABUTMENTS

The U-abutment is a special form of the wing-wall abutment in which the wings are parallel with the roadway or track. The one disadvantage of this type is that a part of the embankment, that outside of the wings, does not receive any protection. This lack of protection may preclude the possibility of using this type where the water rises to a level above the foot of the abutment. On account of the wings being partially buried, they do not receive as much pressure as those of the wing-wall abutment, although experience shows that the wedging action caused by the live load "hammering" the fill between the walls exerts a very considerable pressure. If the side walls are well tied to the face wall with steel reinforcing rods, the thickness of the face wall may be somewhat decreased. Figure 150*a* shows a typical U-abutment.

Figure 150*b* indicates a type in which the side walls are connected by transverse walls. In this way the side walls are made to act as beams of spans equal to the distances between transverse walls, thus reducing the necessary thickness of the same. The floor distributes the loads over a considerable area of soil as well as serving to bring the weight of the filling into use to develop a high degree of stability. The front wall was tied to the side walls with ¾-inch rods. The transverse walls were reinforced with 26 ¾-inch square rods carried into the side walls to within 1 foot of their outer surface. Openings at the bottom of the transverse walls were provided for the convenience of the workmen. Drainage was provided for by pipes, as shown in the illustration.

A very satisfactory way to reduce the quantity of masonry in an abutment is to do away with as much of the earth pressure as possible. Figure 150*c* illustrates a modification of the U-abutment in which this was done by making a solid reinforced-concrete floor on top, which carries the ballast directly, thus allowing the earth to take its natural slope, bringing the toe to the bottom of the front face. In this way all earth pressure except the small amount on the back curtain wall and back

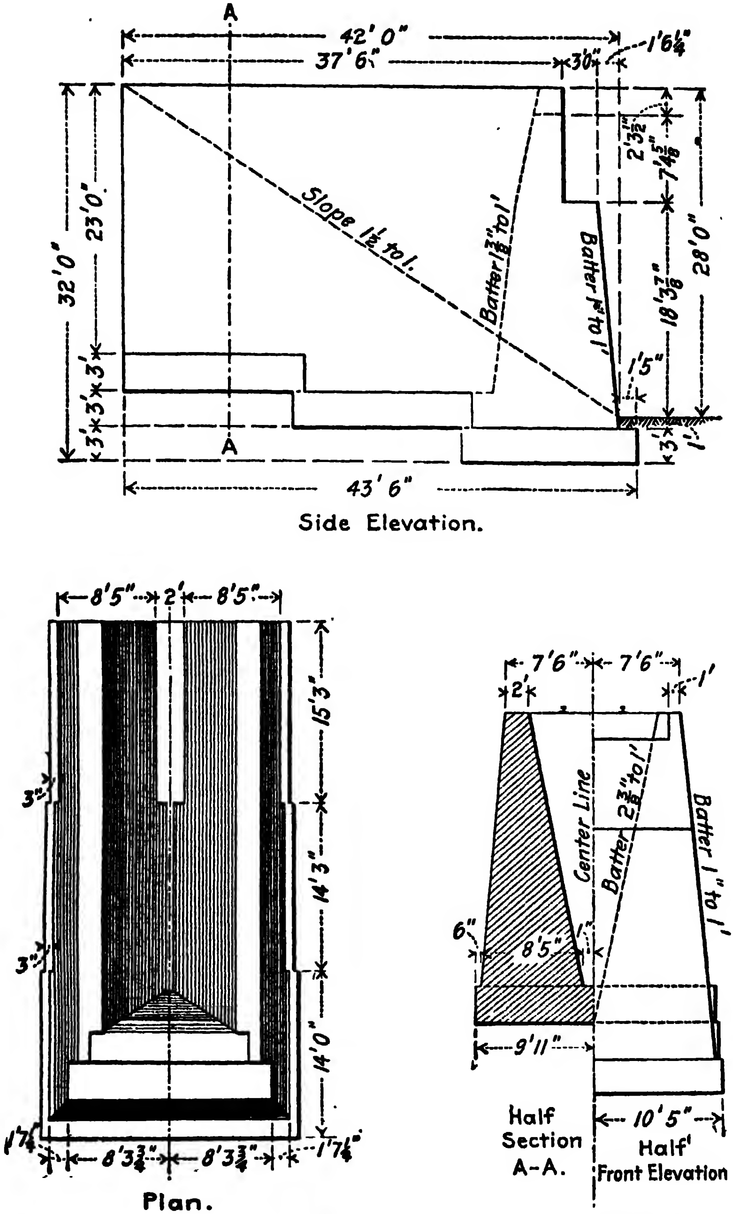


FIG. 150a.—Typical Plain Concrete U-Abutment, Chicago, Milwaukee and St. Paul Railway.

end of the side walls is done away with. The curtain wall is used to insure keeping the outside fill in place.

Figure 150*d* shows the standard type of abutment used on the Harriman Lines for single-track railroad structures. The dimensions of *W*, *D* and *B* for concrete abutments are given in Table No. 150*a* (the dimensions on the diagram being mean values), while the volumes of masonry for different heights and widths are given in Fig. 150*e*. The projection of coping is made 4 inches for stone-masonry abutments 10 feet or under in

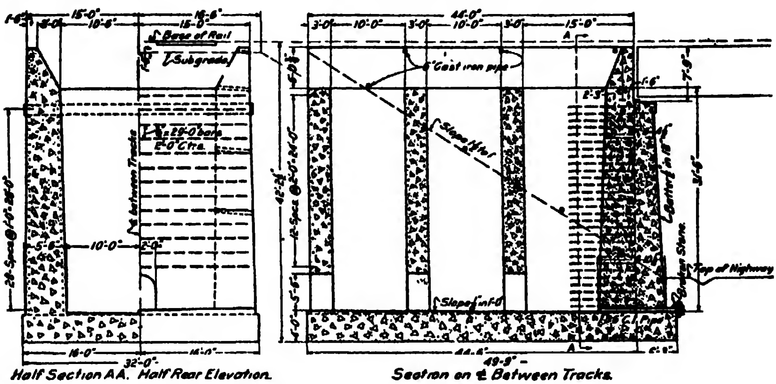


FIG. 150*b*.—Typical U-Abutment for Short Plate Girder Spans, Milwaukee, Sparta, and Northwestern Railway.

height, and 6 inches for heights over 10 feet. All concrete abutments have a coping projection of 4 inches.

Figure 150*f* shows the standard design of U-abutment for steel highway bridges of the Ontario Department of Public Highways. The lengths of wing walls and the values to be added to top widths of face wall to get dimensions *A* and *B* are given below. The yardage of abutment is also given, these values being for an 18-foot roadway.

Height *H*, feet	10	12	14	16	18	20	22
Length wing wall, feet and inches...	9–9	12–9	15–9	18–9	21–9	24–9	27–9
Add to top width for *A*, feet and inches...	0–4	0–8	1–0	1–4	1–8	2–0	2–4
Add to top width for *B*, feet and inches...	1–10	2–2	2–6	2–10	3–2	3–6	3–10
Concrete, cubic yards...	50	69	92	117	147	185	230

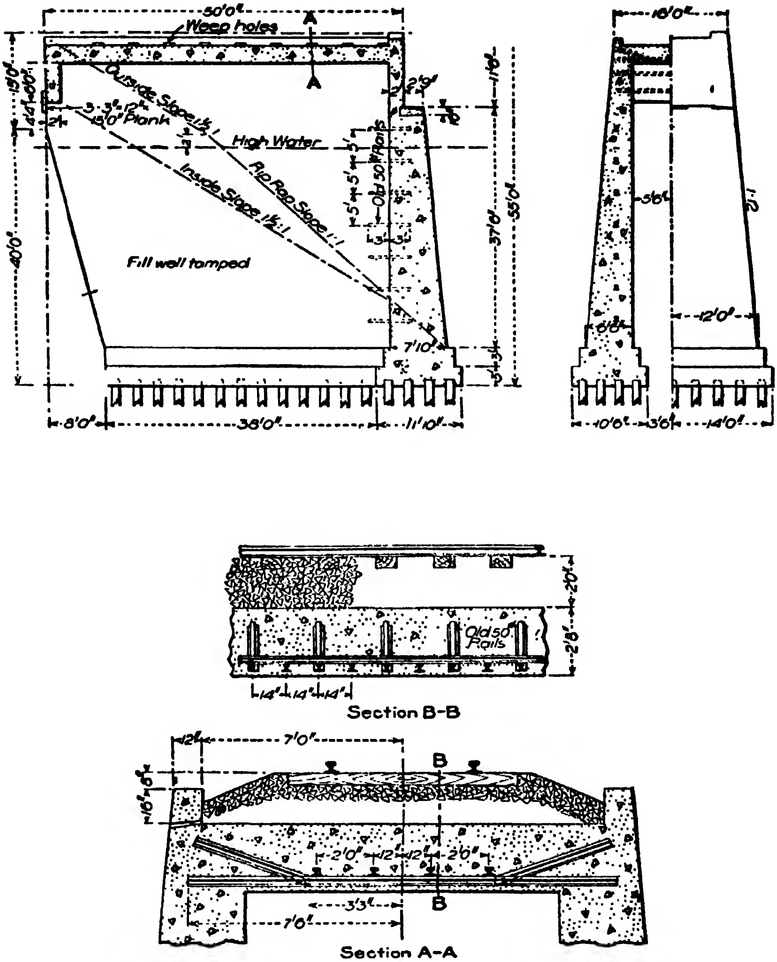


FIG. 150c.—Bridge Abutment with Reinforced-Concrete Deck.

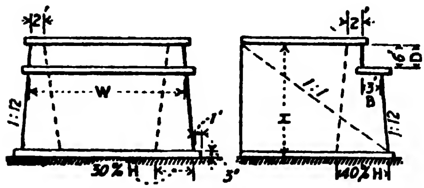


FIG. 150*d*.—Outline of Standard Concrete Abutment.

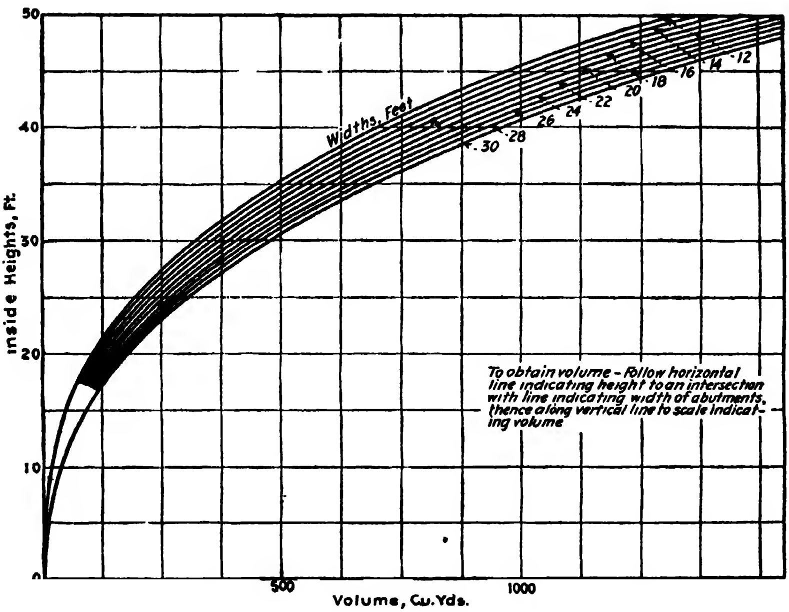


FIG. 150*e*.—Diagram of Cubature of Concrete Abutments.

For a 20-foot roadway the quantities should be increased about 7 cubic yards.

For all heights the thickness of stem at neat line is made not less than four-tenths the height of abutment above neat line, this thickness being obtained, where necessary, by a series of steps along the back face, the number of steps varying from 1

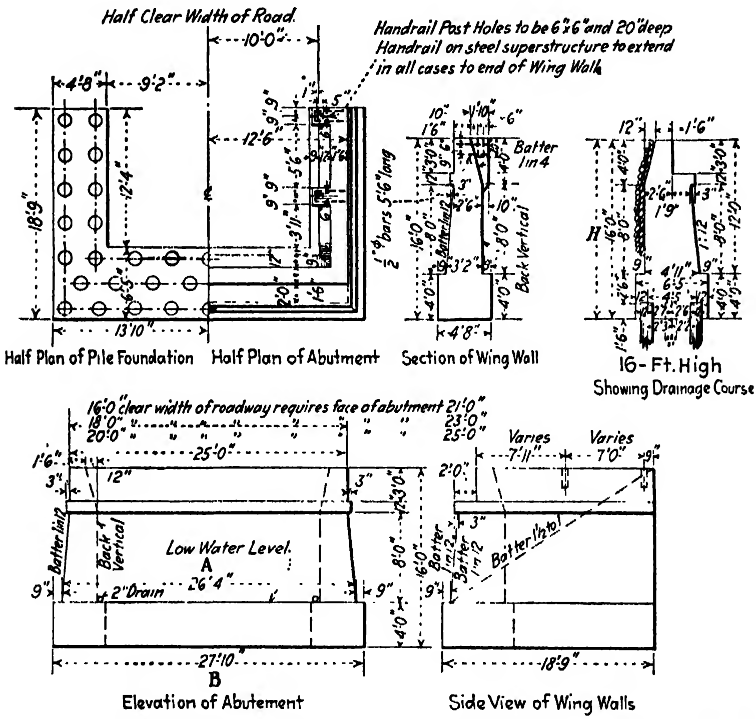
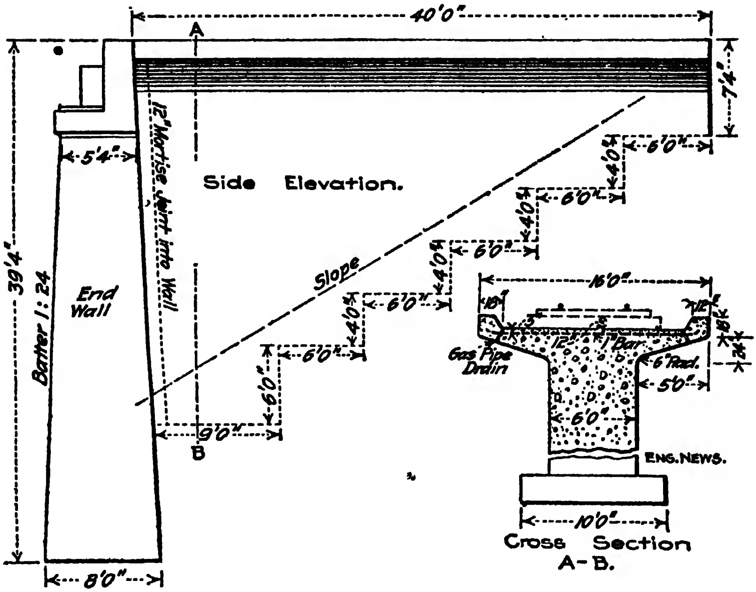


FIG. 150*f*.—Standard Abutment, Ontario Department of Public Highways.

where H is 18 feet to 3 where H is 22 feet. The width of bridge seat and height of back wall varies with the type and length of span.

T-ABUTMENTS.—The T-abutment will usually show some economy over the U-abutment for small heights and narrow roadways. Figure 150g illustrates a skew T-abutment used on the South Bend and Southern Michigan Railway. In this



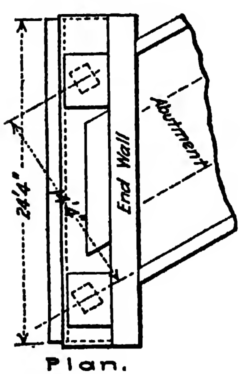


Fig. 150g.—Concrete T-Abutment. South Bend and Southern Michigan Railway over St. Joseph River, near Berrien Springs, Mich.

case the foundation was hardpan and so permitted stepping-up of the base of the stem. Another T-abutment is illustrated in Fig. 150*h*.

TABLE NO. 150*a*

DIMENSIONS OF CONCRETE ABUTMENTS. HARRIMAN LINES' STANDARD, 1906

	Deck plate girders								
Span	20	30	40	50	60	70	80	90	100
W	16– 8	17– 1	17– 2	17– 4½	17– 7	17–10	18– 1	18– 2	18– 2½
D	0– 7	3–0$\frac{1}{16}$	3– 9¾	5–10¾	5–11¾	7– 9¾	9– 3¾	9– 8¾	9–10¾
B	1– 5	2– 1	2– 7	2– 7	2– 1	2– 3	2– 7	2– 8	2– 9
	Through plate girders								
Span	....	30	40	50	60	70	80	90	100
W	....	16– 8	17–10	18– 2	19– 0	19–10	19– 6	19– 8	19–10
D	....	1– 2	1– 2	1– 2	1– 9½	1– 9½	2–10⅝	2–10⅝	2–10⅝
B	....	2– 1	2– 3	2– 3	2– 3	2– 3	2– 7	2– 9	3– 0
	Through riveted trusses					Through pin trusses			
Span	100	110	125	140	150	150	160	180	200
W	20– 0	20– 0	20– 0	20– 8	21– 2	21– 2	21– 2	21– 4	21– 4
D	4– 0¾	4– 0¾	4– 0¾	4– 0¾	4– 0¾	4– 0¾	4– 0¾	4– 0¾	4– 0¾
B	2–11	3– 0	3– 1	3– 3	3– 4	3– 1	3– 1	3– 9	3– 6

Note: All distances are expressed in feet and inches.

ART. 151. BURIED ABUTMENTS

Instead of placing the abutment at the edge of the stream, it is sometimes set back in the embankment and the latter allowed to spill out in front of the same up to the bridge seat. In this case most of the earth pressure back of the abutment will be balanced by that in front, and hence a much less massive abutment is required. On the other hand, a greater length of superstructure is required. In some cases in addition to the buried abutment a pier is placed at the foot of the embankment, in which case the buried abutment, a pier and a short span take the place of the regular abutment.

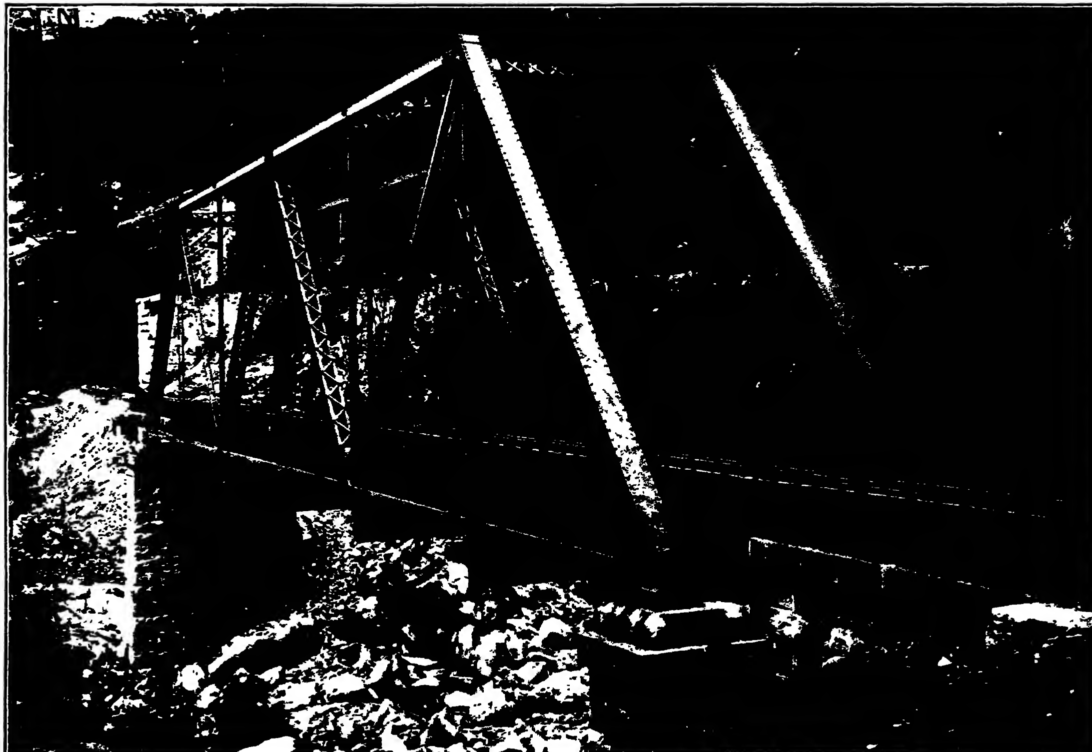


FIG. 150*h*.—View of Single-Track Railroad Bridge showing T-Abutments.

(*Facing p.* 494.)

The buried abutment of the East Haddam bridge across the Connecticut River at East Haddam, Conn., which supports a 99-foot deck span, consists of two reinforced-concrete columns, two footings and a transverse slab, as illustrated in Fig. 151*a*. The footings are tied together with four 1-inch rods encased in

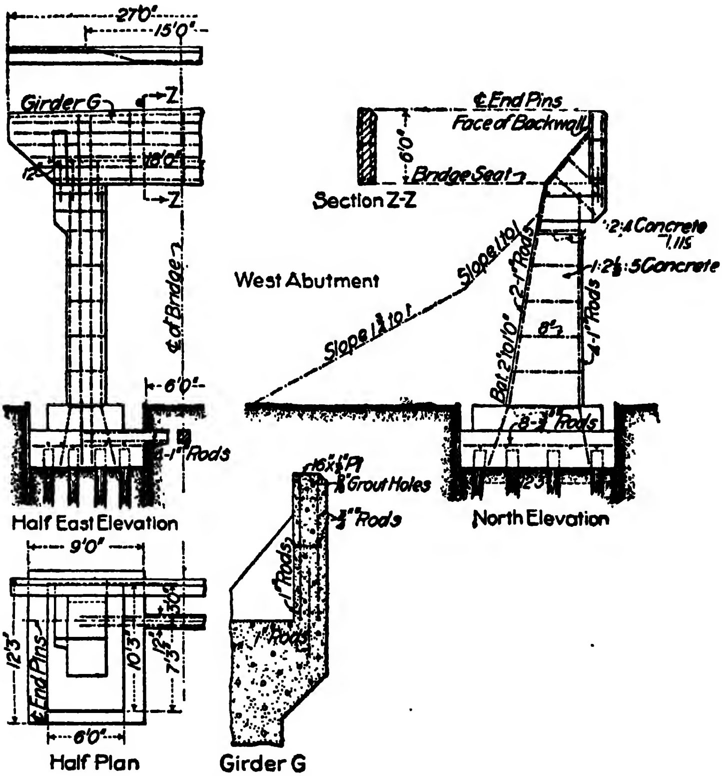


FIG. 151*a*.—A Buried Abutment.

concrete. The transverse reinforced-concrete slab, 15 inches thick and 6 feet deep, connects the tops of the columns and keeps the earth filling from the bridge seats, which rest on the columns. A stone slope pavement protects the bottom of the filling, which is carried up around the columns to within about 6 inches of the bridge seats.

The buried abutment for a bridge on the Louisville and Nashville Railroad is illustrated in Fig. 151*b*. It consists of two shafts; a curtain wall connecting them; a spread footing; bridge seats; parapet and side walls. The shafts are 4 feet thick, 3 feet 11 inches wide at the top and 14 feet wide at the bottom. They are 5 feet apart in the clear and are connected

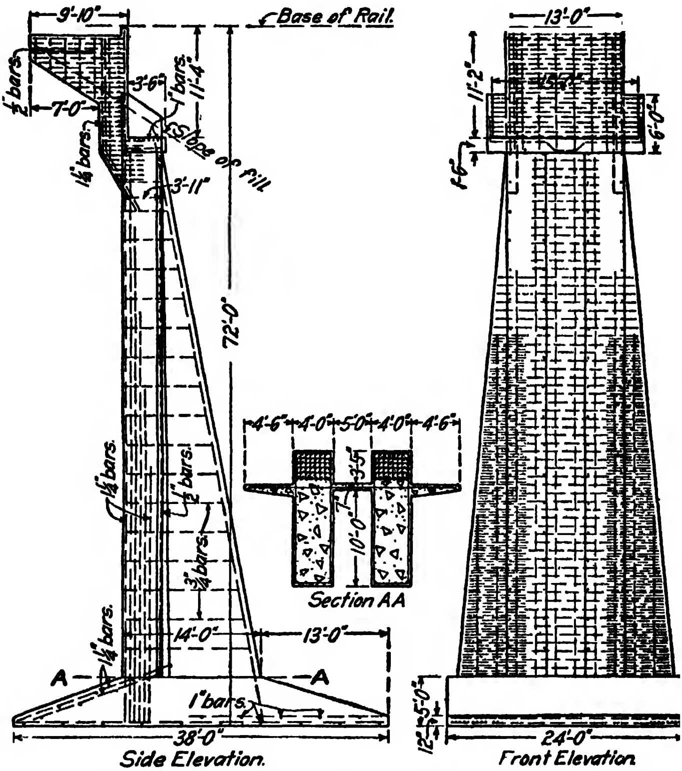


FIG. 151*b*.--Reinforced-Concrete Buried Abutment, Louisville and Nashville Railway Bridge over Cumberland River.

by a 7-inch curtain wall. Wing buttresses, varying from 6 to 15 inches in thickness and from 0 to 4 feet 6 inches in width, stiffen the shafts and transfer some of the load to the footing. The toe end of the footing is reinforced near the lower surface, and the heel end is reinforced near both the lower and upper surfaces. The parapet and side walls are both 13 inches thick. The chief

function of the side walls is to start the embankment slope far enough back to clear the bridge seats. An interesting description of various methods of accomplishing this is given in an article by C. M. LUTHER in Engineering News, vol. 70, page 816, Oct. 23, 1913.

ART. 152. REINFORCED ARCH ABUTMENTS

The reinforced arch abutment may be considered as a modification of the U-abutment. Among the first abutments of this

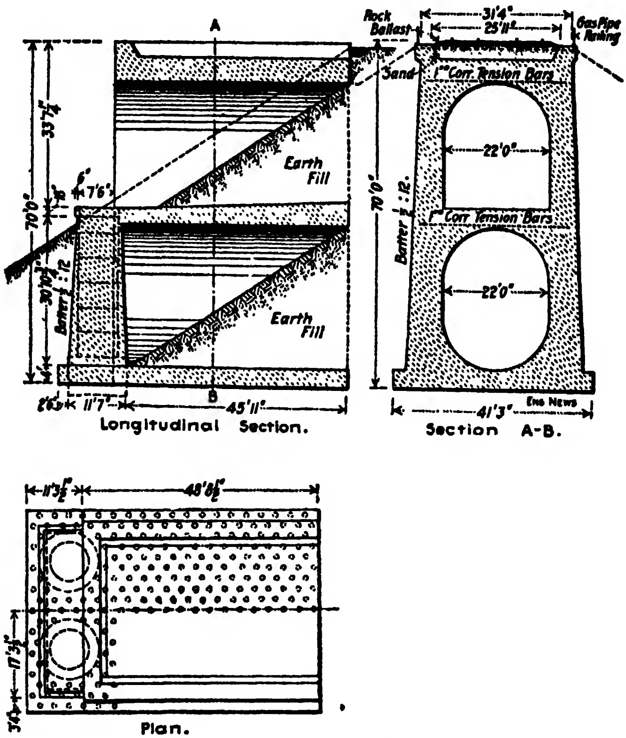


FIG. 152a.—Reinforced-Concrete Arch Abutment.

type were those for a bridge of the Illinois Central Railroad over the Ohio River at Cairo, Ill., designed by W. M. TORRANCE under the direction of F. H. BAINBRIDGE. As shown in Fig. 152a, the abutment consists of face and side walls, an arched

slab at the top, another about halfway down and still another at the bottom. The earth fill is distributed so as to give, in conjunction with the other forces, a nearly uniform bearing over the base of the abutment.

The Chicago, Milwaukee and St. Paul Railway has done much in developing the reinforced arch abutment. Figure 152*b* illustrates the abutments of a bridge near Lombard, Mont. This abutment is said to be more representative of a stage in the development of abutment design than a developed type. Figure 152*c* represents a type which is adopted as a standard on that road. The shafts rest on independent concrete footings and are braced by longitudinal and transverse reinforced-concrete braces. The floor system is composed of slabs and beams, the latter running transversely and carrying their loads to the opposite pairs of shafts and to the longitudinal arched beams at a point midway between the shafts, thus making the beam spacing 8 feet center to center, the shaft spacing longitudinally being 16 feet center to center.

For valuable material on the design and costs of various types of abutments, see a paper by J. H. PRIOR in Proceedings of the American Railway Engineering Association, vol. 13, page 1085, 1912, as well as an article by W. M. TORRANCE on "The Design of High Abutments," in Engineering News, vol. 55, page 36, Jan. 11, 1906.

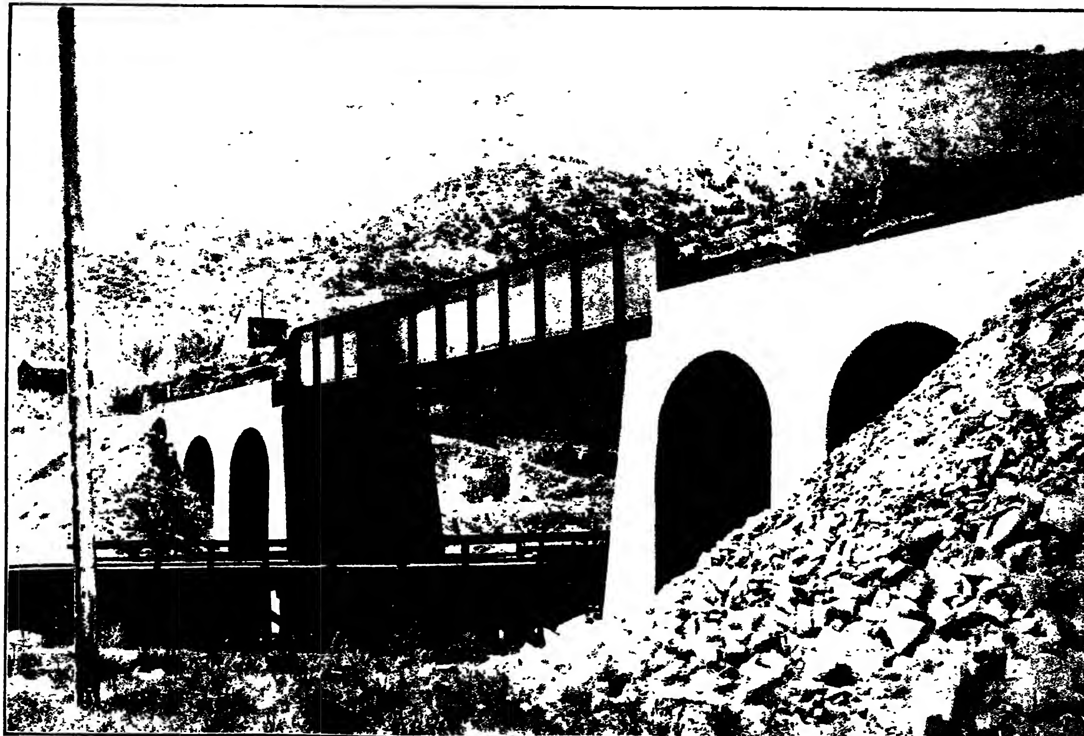


FIG. 152*b*.—Sixteen-Mile Creek Bridge, Chicago, Milwaukee and St. Paul Railway, near Lombard, Mont.

(*Facing p.* 498.)

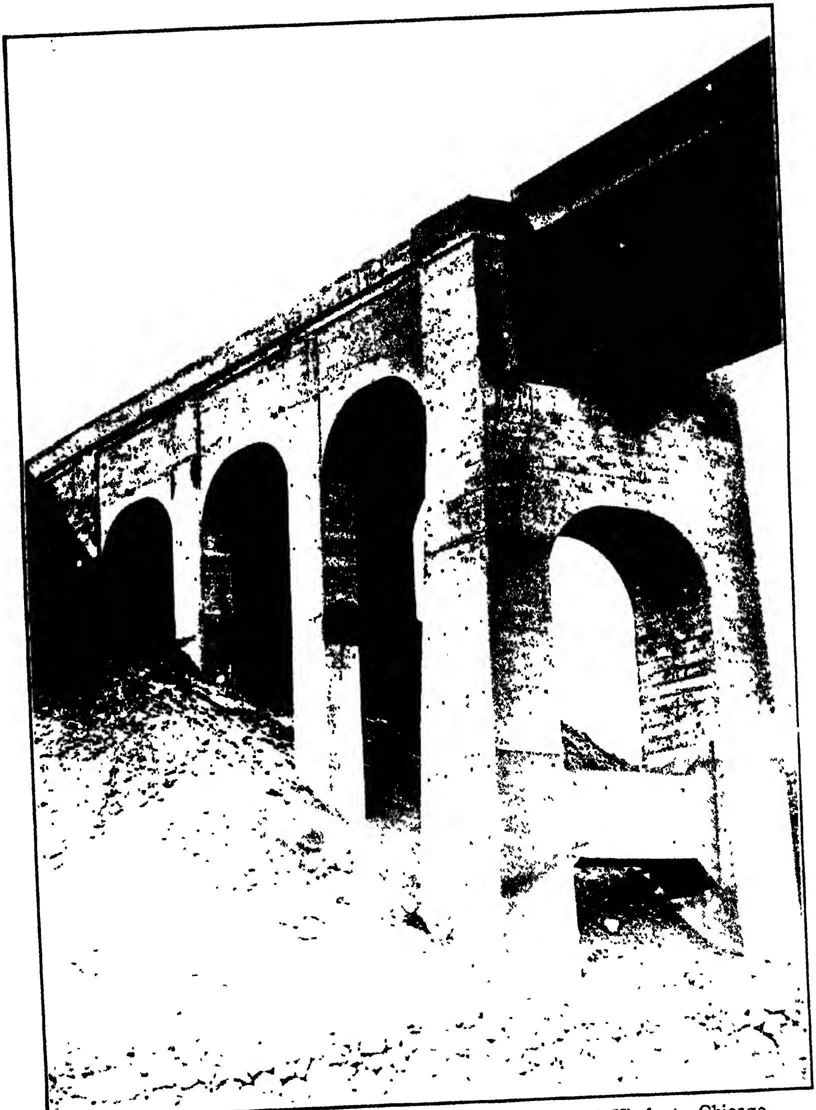


FIG. 152c.—Reinforced-Concrete Arch Abutment of Lind Viaduct, Chicago, Milwaukee, and St. Paul Railway. Built in 1909.

CHAPTER XV

SPREAD FOUNDATIONS

ART. 153. GENERAL CONSIDERATIONS

Foundations for buildings, where bed rock is some distance below the surface, are of three general types: first, those carried deep to rock or hardpan, second, those in which piles are used; and, third, those spread over a given surface. The first type is widely used for heavy buildings where the material overlying the rock is soft, and is exemplified in the pneumatic-caisson process described in Chap. X, and in the open-well process described in Chap. XI. Although the most expensive type of foundation, it offers the advantage of an absolutely unyielding support for the buildings. The subject of bearing piles is treated in Chaps. I to V, inclusive.

The object of the shallow type of foundation is to spread the load over a considerable horizontal area near the surface of the ground; that of pile foundations to distribute the load over a considerable vertical area—the circumferential surface of the piles—as well as carrying some of it to the horizontal stratum at the feet of the piles; while the deep foundation distributes the load over a relatively small area on the rock or hardpan. Where rock is present near the surface there is no foundation problem, it being necessary only to level off the rock with a layer of concrete and place the columns of walls directly upon it, although a spread footing may be used where the foundation loads are very heavy.

In many localities the most common type for light buildings is the shallow foundation, and in modern development it is being used to a considerable extent for heavy structures. In its original and simplest form the shallow foundation consists of a wide concrete or masonry footing with its maximum area at

the base and stepped off to decrease in horizontal area toward the top, the latter being of sufficient size to form a seat for the wall or column base. Although this makes a satisfactory footing for small loads, it is not well adapted to heavy loads, owing to the depth required to get the necessary spread of base. Other forms of shallow foundations have been developed, such as the wooden grillage, the inverted arch, the steel I-beam grillage and the reinforced-concrete spread footing, all of which require less depth.

The shallow type of foundation is relatively inexpensive, and easily and quickly constructed, but it possesses the disadvantage of failing to furnish a rigid and unyielding support for the building. Where founded on compact sand, the settlement will be slight, seldom more than ½ inch, but where founded on a material like the Chicago clay the settlement may in time amount to 2 feet or more. Hence, heavy buildings resting on shallow foundations are built to allow for a certain amount of settlement, or else the foundations are so constructed that powerful hydraulic jacks can be used to raise the building to permit shimming up. Uniform settlement causes but little trouble and can be easily taken care of; but unequal settlement causes the walls to crack. The most satisfactory method of guarding against unequal settlement was early found to be the use of independent footings for the columns, the area of the base of each footing being so proportioned that the unit-pressure is the same under all footings.

ART. 154. EARLY TYPES OF FOOTINGS

MASONRY FOOTINGS.—This type, which was one of the earliest, is still the standard for light loads. It may be built of concrete, brick masonry or stone masonry, the first mentioned being the most widely used at present. In designing the footing, the area of the base is found by dividing the wall load by the safe bearing power of the soil as given in Art. 184. To safeguard the masonry against crushing, the compressive unit-stress on any horizontal section should not exceed the values

given in Table No. 154*a*. The top of the footing is made a little larger than the column base or wall.

Having determined the top and bottom areas of the footing, the next step is to design the offsets, which fix the depth of the footing. As usually designed, these offsets are assumed to act as free cantilevers, and so the allowable offset of any section will depend upon: first, the pressure on the under side; second, the transverse strength of the masonry; and, third, the thickness of the course. The center of gravity of the base should coincide with the axis of the load; otherwise, additional stresses will develop.

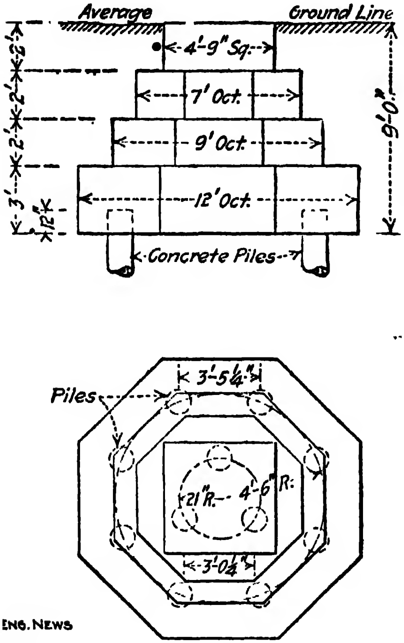


FIG. 154*a*.—A Typical Masonry Footing.

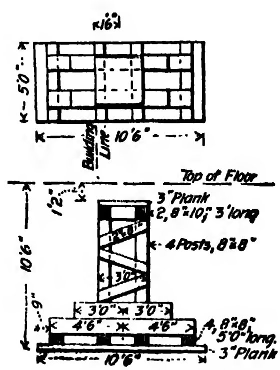


FIG. 154*b*.—Spread Footing of Timber Column.

TABLE NO. 154*a*

CHARACTER OF MASONRY	SAFE COMPRESSION, POUNDS PER SQUARE INCH
Common brick, hard-burned (portland cement mortar).....	200
Common brick, ordinary (portland cement mortar).........	175
Rubble masonry, uncoursed (portland cement mortar)......	200
Rubble masonry, coursed (portland cement mortar)........	250
Portland cement concrete, 1-2-4 mixture..................	550
Portland cement concrete, 1-2½-5 mixture................	450
Portland cement concrete, 1-3-6 mixture..................	350

Considering the case of a footing for the wall of a building, let p denote the unit-pressure in pounds per square foot on the bottom of the course in question; R the modulus of rupture of the masonry; f the factor of safety used; t the thickness of the course in inches; and o the allowable offset of the course in inches. The following formula is then obtained:

$$o = t\sqrt{\frac{48R}{pf}}.$$

A factor of safety of about 6 will usually be advisable. In designing masonry footings for columns, the method given in Art. 162 is recommended, although the above formula may give sufficient precision.

OTHER EARLY TYPES OF FOOTINGS.—Owing to its lack of transverse strength, masonry is ill adapted to take loads which cause flexural stresses of any magnitude. For this reason various substitutes have been adopted, the idea being to use some material having considerable transverse strength in order to reduce the necessary depth.

Among the early types was the timber grillage. This consists of two or more layers of heavy timbers, each layer being placed at right angles to the one above and below, the top and bottom being often sheathed with a layer of planking. The various courses are well tied together with drift bolts.

Examples of such grillages have been dug up after being buried from 50 to 100 years, and where below ground-water level have been found to be in a perfect state of preservation. The high price of timber, together with its relatively low transverse strength and the uncertainty of the future ground-water level, makes timber an undesirable material for use in permanent foundations. For temporary structures, such as exposition buildings, it is still used to some extent. Figures 154b and 154c show the details of such a grillage when used under columns.

Another type, which was employed in some of the early heavy Chicago buildings, consisted of a thick concrete platform continuous over the whole area of the building site, forming a

deep monolithic slab at the cellar-floor level and on which the columns and walls rested. The effect of variation in the magni-

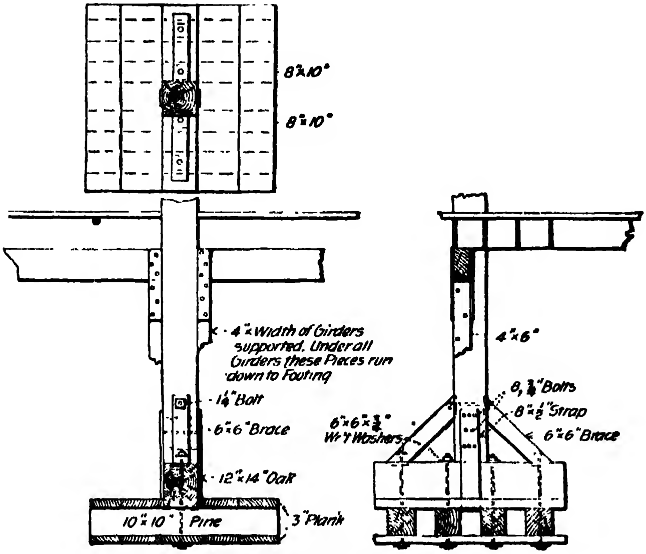


Fig. 154c.—Braced Column Footing.

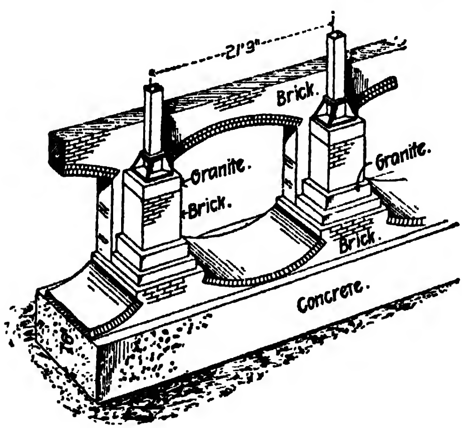


Fig. 154d.—Footing of Inverted Masonry Arch, Drexel Building, Philadelphia, Pa.

tude of the concentrated loads was to crack the concrete bed into a number of independent footings and this was naturally fol-

lowed by great irregularity in the settlement of various parts of the building. Hence this form of footing was never entirely satisfactory.

Another early type of spread foundation was that of the inverted masonry arch which was first used in the Drexel Building, Philadelphia, Pa., built in 1893, the details of which are shown in Fig. 154*d*. Another notable example of the use of this type was in the World Building, New York City. Both of these structures were among the early examples of the modern steel office building. In the Drexel Building the arches were made of brick, which distributed the column loads through continuous lines of concrete bases in the column rows, on the soil below. Although the brick-masonry arch is no longer used, the principle is still employed in the reinforced-concrete arch footing described in Art. 163.

ART. 155. MODERN TYPES OF SPREAD FOUNDATIONS

The two modern types of spread foundations are the steel I-beam grillage and the reinforced-concrete spread footing. The steel I-beam grillage dates back to the types described in the preceding article; but since it is still a standard type it is here described. The conditions surrounding its development are as follows: In the business district of Chicago the soil conditions are peculiar, made ground extending to a depth of about 14 feet below street grade while below that occurs a stratum of hard stiff clay 6 to 12 feet thick. Below this the clay, while having the same general characteristics as that above, becomes softer and remains so to a depth of 75 feet or more. The upper stiff clay makes a first-class foundation bed, but the softer clay below offers little supporting power.

After the great Chicago fire most of the new buildings were founded on masonry footings which rested on this hard clay stratum. Owing to the rapid increase in the size and weight of buildings it became necessary to increase the area of the base of footings and this in turn compelled the use of deeper footings. As the bearing power of the soft clay below the hard stratum was small, the only practicable method of obtaining this greater

depth was to extend the footing up into the cellar; and thus the cellars soon became filled with pyramids of masonry, robbing them of valuable space. This, together with the fact that the masonry footings, on account of their large mass, formed too large a proportion of the total load and were expensive, started the search for a better type of footing for heavy loads. The type thus developed consisted of crossed layers of old steel rails, which were soon superseded by steel I-beams, both shapes being thoroughly embedded in concrete as a protection against rust.

Probably the first building in America to be built on a steel grillage foundation was the Montauk Block, Chicago, built in 1878, and designed by Burnham and Root, architects. The ordinary masonry footing was used for a part of the building, but to obtain space for the boiler a grillage of steel rails embedded in concrete was used in one part of the cellar. Soon after this date the price of steel I-beams dropped sufficiently to make them available for this purpose. On account of their larger section modulus for any given weight per foot, they are much more economical than rails, and in a short time I-beams were adopted exclusively. For very heavy loads built-up girders are often used in place of I-beams.

In the construction of I-beam grillages two or more tiers are used, the exact number depending on the desired spread of base. Each tier is placed at right angles to the one below it and the load is carried to the soil through beam action. The individual beams of each tier should be held in place by cast-iron or gas-pipe separators, preferably the former. These separators should be placed near each end of the beams and at intermediate positions not over 5 feet apart. The beams should be spaced so as to give a clearance of not less than 3 inches, in order that the concrete may readily be filled in between the beams; and not more than one and one-half times the width of the flange, in order to reduce the stresses in the concrete filling. The latter requirement cannot always be met.

Concrete should be filled in between the beams and also placed around the sides, top and bottom of the grillage. The

thickness of the bottom layer should not be less than 12 inches, and the top and sides should have a protective coating of at least 4 inches net thickness. If portland cement concrete is used, the mixture should not be leaner than 1-3-6. A layer of cement grout of from ½ to 1 inch in thickness should be placed between the tiers of beams.

ART. 156. DESIGNING LOADS FOR SPREAD FOOTINGS

The loads coming to the basement columns are dead, live and wind. Dead loads, which include the weight of the frame and floors of the building, can be closely figured. Live and wind are assumed loads, and in most cities are governed by the building codes, which differ considerably. For office buildings a widely used specification calls for a basement column loading of 50 pounds per square foot of roof, 75 pounds per square foot on the top floor, 75 pounds less 5 percent for the floor below, with a 5 percent increase in reduction for each successive floor below until 50 percent is reached, which value is used for all lower floors. The New York code (1920) calls for 40 pounds per square foot for the roof and 60 pounds for the top floor, with the same reductions for lower floors as given above.

To determine the area of the footing, the live load is often reduced from that used in designing columns. One widely used specification is as follows: for warehouses and factories, the full live load; for stores, buildings for light manufacturing purposes, churches, schoolhouses and places of public amusement, 75 percent; and for hotels, dwellings, apartment houses, etc., 60 percent. The New York code (1920) specifies that full live load as given above shall be used in figuring loads on footings.

In spread foundations a moderate settlement is to be expected, but care must be exercised to make this uniform. The problem is to design the footings so that in no case shall the movement be excessive and so that all footings settle the same amount. However, uniform settlement is hard to get where the percentage of live to total load varies considerably for the differ-

ent footings. Dead load usually causes the greater part of the total settlement, as it is the first load to be applied and remains a constant value. The actual live load is usually much less than the assumed value, and even this, in most buildings, comes on only at intervals. Differential settlement during the erection of steel-frame buildings is very troublesome on account of the difficulties in fitting together the various members. For these reasons it is customary to design footings for a uniform pressure per square foot under dead load or under dead plus *partial* live. Many building codes omit wind in the design of footings and except for high, narrow buildings this would seem to be good practice.

The following formulas have been used to get footing areas:

$$\text{McCullough } A = \frac{DT_1}{BD_1}$$

$$\text{Schneider } A = \frac{DT_2}{BD_2}$$

$$\text{Moran } A = \frac{HT_2}{BH_1}$$

$$\text{Fleming } A = \frac{KT_2}{BK_1}$$

$$\text{Live} + \text{Dead } A = \frac{T}{B}$$

in which A denotes the area of the footing, D the dead load, D_1 the dead load on footing with smallest ratio of live to dead load, D_2 the dead load on footing with largest ratio of live to dead load, T the total load on footing, T_1 the total load on footing with smallest ratio of live to dead load, T_2 the total load on footing with largest ratio of live to dead load, H the dead load plus one-half the probable live load plus one-half the probable wind load, H_1 the dead load plus one-half the probable live plus one-half the probable wind on footing with largest ratio of live to dead load, K dead load plus one-third live load, K_1 dead plus one-third live load on footing with largest ratio of live to dead load, and B the allowable pressure on the soil.

The McCullough and Schneider formulas give uniform soil pressure under dead loads, while the Moran and Fleming formulas give the same unit-pressure for all columns under dead plus partial live and wind and dead plus partial live, respectively. In the McCullough method of design, that footing receiving the smallest ratio of live to dead load is selected and proportioned for the combined live and dead loads. The dead load on this footing is then divided by the area of the footing and this reduced pressure is divided into the dead loads on the other footings to get their respective areas.

By the Schneider method, that footing receiving the largest ratio of live to dead load is selected and proportioned for the combined live and dead loads. The area thus found is divided into the dead load to get the unit pressure that is to be used for all the other footings, this unit pressure being divided into the dead loads of the other footings to get their required areas.

By the Moran method of proportioning, that footing receiving the largest ratio of live plus wind load to dead load is proportioned for the combined live, wind and dead loads. The dead plus one-half probable[1] live plus one-half probable wind load on this footing, is then divided by the area of the footing, and this value is then divided into the dead plus one-half probable live plus one-half probable wind loads of the other footings to get their areas.

The Fleming formula is the same as the Moran except that wind load is not considered unless it exceeds one-half the sum

[1] Owing to the impulse effect of wind loads and the inertia of the building, the full theoretical wind load never reaches the footings and so the probable wind load may be taken at 50 per cent of the maximum. The maximum probable live load is the load which in the opinion of the designer will actually come upon the footings, and is to be determined by a study of the conditions which will obtain when the building is occupied. For instance, in a schoolhouse the number of children in each classroom and the weight of desks, chairs, etc. may be determined with considerable accuracy and these loads will make the maximum probable live load. As a further illustration, in many schoolhouses there is an assembly room, which is only used when the classrooms are vacant, and consequently if classroom loads are used assembly-room loads should be omitted, or *vice versa;* the greater one of these loadings to be used for the probable load. A full explanation of his method may also be found in the revised edition of Kidder's Architects' and Builders' Pocket Book.

TABLE 156a

		Fleming	McCullough	Schneider	Moran	Live plus dead load		
Column 44	Area, square feet	87.4	67.6	103.6	90.7	67.6	Total load	= 473,000 pounds
	Dead plus live load	5,410	7,000	4,550	5,220	7,000	Dead load	= 435,000 pounds
	Dead plus one-third live load	5,130	6,650	4,330	4,940	6,630	Live load	= 38,000 pounds
	Dead plus one-half probable live	5,080	6,590	4,290	4,900	6,570	Dead + 1/3 live load	= 447,700 pounds
	Dead load	4,980	6,440	4,200	4,800	6,440	Dead + 1/2 probable live	= 444,500 pounds
							Dead load	= 92 percent
							Live load	= 8 percent
Column 1	Area, square feet	89.4	58.3	89.3	88.4	86.4	Total load	= 605,000 pounds
	Dead plus live load	6,770	10,380	6,770	6,840	7,000	Dead load	= 375,000 pounds
	Dead plus one-third live load	5,130	7,870	5,130	5,180	5,230	Live load	= 230,000 pounds
	Dead plus one-half probable live	4,840	7,420	4,840	4,900	5,000	Dead + 1/3 live load	= 457,700 pounds
	Dead load	4,190	6,440	4,200	4,240	4,340	Dead + 1/2 probable live	= 432,500 pounds
							Dead load	= 62 percent
							Live load	= 38 percent
Column 29	Area, square feet	94.0	66.6	102.2	95.7	84.0	Total load	= 588,000 pounds
	Dead plus live load	6,260	8,820	5,750	6,140	7,000	Dead load	= 429,000 pounds
	Dead plus one-third live load	5,130	7,230	4,720	5,030	5,740	Live load	= 159,000 pounds
	Dead plus one-half probable live	5,100	7,050	4,590	4,900	5,590	Dead + 1/3 live load	= 482,000 pounds
	Dead load	4,570	6,440	4,200	4,480	5,110	Dead + 1/2 probable live	= 468,800 pounds
							Dead load	= 73 percent
							Live load	= 27 percent
Column 24	Area, square feet	90.3	58.9	90.3	90.3	90.3	Total load	= 632,000 pounds
	Dead plus live load	7,000	10,740	7,000	7,000	7,000	Dead load	= 379,000 pounds
	Dead plus one-third live load	5,130	7,860	5,130	5,130	5,130	Live load	= 253,000 pounds
	Dead plus one-half probable live	4,900	7,500	4,910	4,900	4,900	Dead + 1/3 live load	= 463,300 pounds
	Dead load	4,200	6,440	4,200	4,200	4,200	Dead + 1/2 probable live	= 442,300 pounds
							Dead load	= 60 percent
							Live load	= 40 percent

The allowable unit-bearing on the ground is taken at 7000 pounds per square foot. In this example the probable live load is taken as one-half the full live load.

of the dead and live loads and the footings are designed for uniform pressure under dead plus one-third live load.

In Table No. 156*a* the results are tabulated for the design of the footings of an actual structure by each of the five formulas. In this example the Moran formula is used omitting wind and assuming the probable live load to be one-half the maximum live.

The McCULLOUGH formula is unsatisfactory because of the high stresses under some of the columns, as column 24. The SCHNEIDER formula, which also gives under dead load the same unit pressures for all columns, is very conservative. Neither of these formulas gives any consideration to the effect of live load in causing settlement. The MORAN and FLEMING formulas seem better in this respect. Where the probable live load can be closely ascertained, the Moran formula is recommended, but where conditions are such as to make the probable live load uncertain, the FLEMING formula may be used. The dead plus live load formula gives entirely too much weight to live load.

For complete discussions of this subject see Engineering News, vol. 69, page 463, Mar. 6, 1913, and Engineering News-Record, vol. 85, page 219, July 29, 1920.

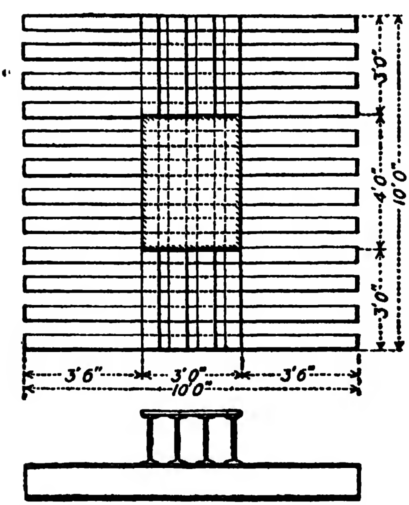


FIG. 157*a*.—Steel I-Beam Grillage for a Single Column.

ART. 157. DESIGN OF I-BEAM GRILLAGES

In designing steel grillage foundations the following assumptions are made: first, the pressure from the footing is uniformly distributed over the bed; second, the pressure of one tier of beams on another is uniformly distributed over the latter; third, each tier acts independently of all other tiers; and fourth, the concrete filling

and covering carries no stress, acting merely as a protection against corrosion.

For the single-column grillage the square base is the most economical shape. Where the possible width is restricted, as in wall-column footings, the grillage should be made as nearly square as possible. Economy also results in using a minimum number of tiers.

EXAMPLE OF DESIGN OF SINGLE-COLUMN FOOTING.—Load = 600,000 pounds. Allowable pressure on foundation bed = 6000 pounds per square foot. Size of column base = 3 by 4 ft. Required area of base = 600,000 /6000 = 100 square feet. A base 10 feet square is adopted. Assume two tiers of beams. For the top tier, the maximum bending moment $M = \frac{600,000}{2} \times \frac{5-2}{2} \times 12 = 5,400,000$ pound-inches. Using 16,000 pounds per square inch as the safe unit-stress in the outer fiber, the total section modulus required $= I/e = 5,400,000/16,000 \doteq 337$ inch^3. Trying various combinations of beams, the following results are obtained:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	3	112.3	20	65.4	116.9	6.25	8.6
2	4	84.2	18	54.7	88.4	6.00	4.0
3	5	67.4	15	55	67 8	5.75	1.8

The choice lies between Nos. 1 and 2, since No. 3 does not give sufficient clearance. The weight favors No. 1, being 250 pounds lighter, while No. 2 gives a more satisfactory clearance and has less depth, thus saving on concrete filling and also excavation.

For the lowerer tier: Maximum $M = \frac{600,000}{2} \times \frac{5-1.5}{2} \times 12 =$ 6,300,000 pound-inches. Total required $I/e = 6,300,000/16,000 = 394$ inch^3. The following results are obtained by trying various combinations of beams:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	10	39.4	12	40.8	44 8	5.25	7.5
2	12	32.9	12	31.8	36.0	5 00	5.5
3	14	28.2	12	31.8	36.0	5.00	3.8
4	16	24.6	10	25.4	24.4	4.66	3.0

The choice lies between Nos. 2 and 4; the latter has 220 pounds more steel, but the clearance is better and a 2-inch depth of concrete is saved.

After designing for bending, the beams should be checked for shearing and buckling of the web. The maximum shear for the upper tier of beams is $600{,}000 \times 3/10 = 180{,}000$ pounds. By the Carnegie handbook the safe shearing strength of four 18-inch 55-pound beams is $82{,}800 \times 4 = 331{,}200$ pounds. The maximum shear in the lower tier is $600{,}000 \times 3.5/10 = 210{,}000$ pounds, while the safe shearing strength of 12 12-inch $31\frac{1}{2}$-pound beams is $42{,}000 \times 12 = 504{,}000$ pounds.

The maximum buckling stress may be considered to occur on a length equal to the length over which the superimposed load is distributed plus one-half the depth of the beam. On this basis, the unit buckling stress for the upper tier of beams is $\dfrac{600{,}000}{(48 + 9) \times 4 \times 0.46} = 5730$ pounds per square inch. By the Carnegie handbook the safe unit-stress is 12,200 pounds per square inch. Likewise, for the lower tier the unit buckling stress is $\dfrac{600{,}000}{(36 + 6) \times 12 \times 0.35} = 3400$ pounds, while the safe unit-stress is 13,060.

ART. 158. DESIGN OF TWO- AND THREE-COLUMN FOOTINGS

Where the two-column loads are equal, the base of the footing should be rectangular in shape and symmetrical about a line

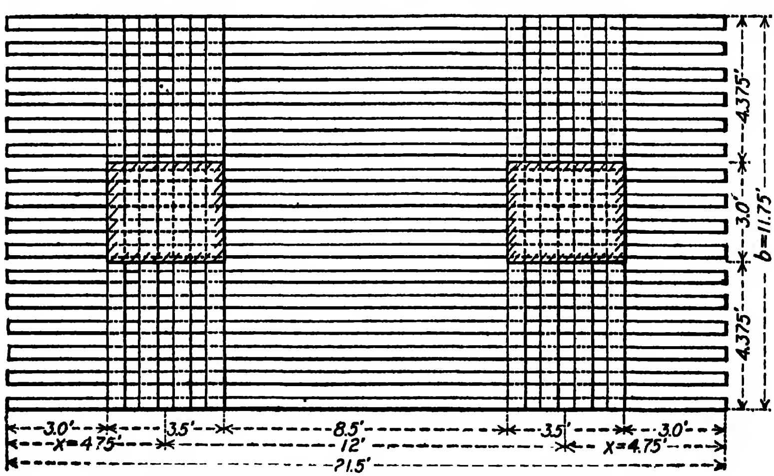


FIG. 158a.—Double-Column Footing of Steel I-Beams.

midway between the columns. When the total area of the base has been determined and the distance between columns

fixed, the proportion of length to breadth for the base of footing should be such that the moment in the lower tier of beams under the column centers equals that at a point midway between the columns. This makes the three maximum moments approximately equal, and gives the greatest economy of material.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, EQUAL LOADS.—Column loads = 500,000 pounds. Column spacing = 12 feet. Allowable pressure on ground = 4000 pounds per square foot. Size of column bases = $3\frac{1}{2}$ by 3 feet. Allowable unit-stress in beams = 16,000 pounds per square inch. To get the value x that will make the three moments equal, $500{,}000(6 - x)/2 = 500{,}000x^2/2(6 + x) - 500{,}000\,(\frac{7}{16})$, whence x = 4.77 ft. Required bearing area of base = 1,000,000 /4000 = 250 sq. ft. Using a value of x of 4.75 feet, $b = 250/(12 + 2 \times 4.75) = 11.63$ feet; say 11.75. Let two tiers of beams be assumed. Computing for top tier: Maximum $M = 500{,}000(11.75 - 3)\ 12/8 = 6{,}560{,}000$ pound-inches. Total required $I/e = 6{,}560{,}000/16{,}000 = 410$ inch³. After trying various combinations of beams, the results are:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	3	136.7	24	79.9	173.9	7.0	10.5
2	4	102.5	20	65.4	116.9	6 25	5.3
3	5	82.0	18	54.7	88.4	6.0	3.0

No. 2 will be adopted.

For lower tier the three positions of maximum bending moment are at the center and 4.45 feet from each end. M at center = 500,000 $(6 - 5.375)12 = 3{,}750{,}000$ pound-inches. M at 4.45 feet from the end =

$$\left[\frac{500{,}000}{10.75}\cdot\frac{4.45^2}{2} - \frac{500{,}000}{3.5}\cdot\frac{1.45^2}{2}\right]12 = 3{,}720{,}000 \text{ pound-inches.}$$

Total required $I/e = 3{,}720{,}000/16{,}000 = 234$. Upon trying various combinations of beams, the results are found to be:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	12	19 5	9	21.8	18.9	4.33	8.1
2	14	16.7	9	21.8	18.9	4 33	6.1
3	16	14.6	8	18.4	14.2	4.0	5.1
4	18	13.0	8	18.4	14.2	4.0	4.1

No. 3 will be adopted,

When the column loads are not equal, the center of gravity of the base of the grillage is usually made to coincide with the line of action of the resultant of the two column loads by making the base a trapezoid; or, if the loads are nearly equal, it may be done by using a rectangular shape and making the cantilever end at the heavy load longer than the other cantilever end.

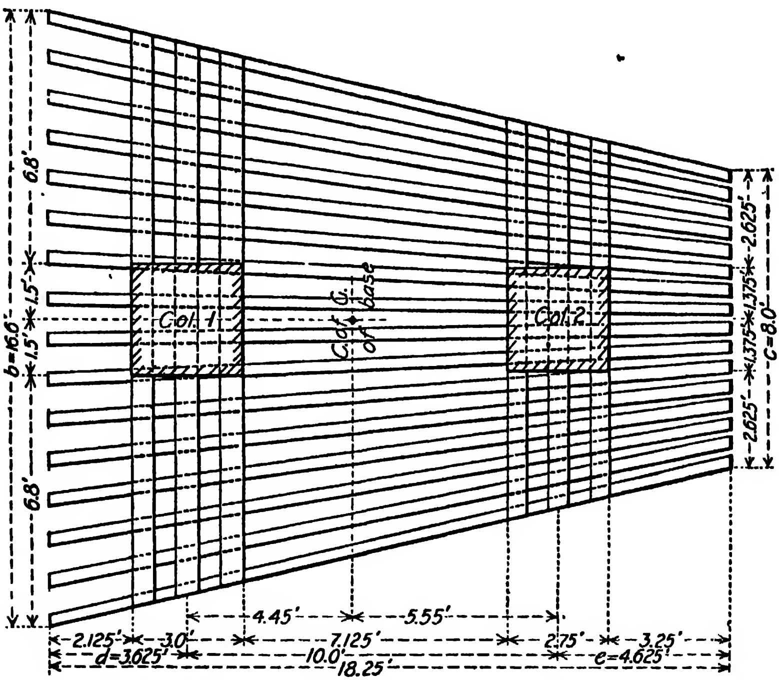


FIG. 158b.—Steel I-Beam Grillage for Two Columns Supporting Unequal Loads. The load on column 1 is 500,000 pounds; that on column 2 is 400,000 pounds.

The trapezoidal shape may be obtained either by using a larger number of beams at the heavy load end, or by using the same number of beams and spacing them more closely at one end than at the other. A combination of the two methods is sometimes used.

If the proportions of the base are so fixed that the bending moment under the center of each column equals that at the

center of gravity of the base, the three maximum moments in the lower tier of grillage will be closely equal; this condition gives approximately the minimum amount of material. The most satisfactory method of determining the dimensions to secure this result is by trial.

EXAMPLE OF DESIGN OF DOUBLE-COLUMN FOOTING, UNEQUAL LOADS.— Column loads and spacing as shown in Fig. 158*b*. Allowable pressure on foundation bed = 4000 pounds per square foot. Size of column bases as shown in Fig. 158*b*. Allowable unit-stress in beams = 16,000 pounds per square inch. Required bearing area of base = 900,000/4000 = 225 square feet. Distance from column 1 to resultant of both column loads = 400,000 × 10/900,000 = 4.45 feet.

Assuming $d = 3.625$ and $e = 4.625$, the values of b and c are given by the formula $\frac{A}{L}\left(1 + 6\frac{f}{L}\right)$, the plus sign for b and the minus for c, where A denotes the area of the base, L the length of the base and f the distance from the center of the base to the center of gravity of the loads.

Substituting in this formula, $b = \frac{225}{18.25}\left(1 + \frac{6 \times 1.05}{18.25}\right) = 16.6$ feet and $c = \frac{225}{18.25}\left(1 - \frac{6 \times 1.05}{18.25}\right) = 8.0$ feet.

Letting y_1 denote the breadth of the footing at the center of column 1, $y_1 = c + (b - c)(e + g)/L = 8 + 8.6 \times 14.62/18.25 = 14.88$ feet.

The moment under the center of column 1 is $4000(2b + y_1)d^2/6 - 500{,}000 \times 3/8 = (2 \times 16.6 + 14.88)\ 3.625^2/6 - 187{,}500 = 233{,}700$ foot-pounds.

Letting y_3 denote the breadth of footing at center of gravity, $y = c + (b - c)(e + k)/L = 8 + 8.6 \times 10.17/18.25 = 12.8$ feet.

The moment at the center of gravity of the loads is
$4000(2b + y_3)(d + h)^2/6 - 500{,}000 \times 4.45 = (2 \times 16.6 + 12.8)8.07^2/6 - 2{,}225{,}000 = -225{,}000$ foot-pounds.

Letting y_2 denote the breadth of the footing at the center of column 2, $y_2 = c + (b - c)c/L = 8 + 8.6 \times 4.62/18.25 = 10.18$ feet.

The moment under the center of column 2 is $4000(y_2 + 2c)e^2/6 - 400{,}000 \times 2.75/8 = 4000(10.18 + 2 \times 8)4.62^2/6 - 137{,}500 = 236{,}000$ foot-pounds.

These moments, being nearly equal, show that d and e were correctly assumed.

Using two tiers of beams, the computations for the upper tier under column 1 give:

Maximum $M = (500{,}000/8)(14.88 - 3)12 = 8{,}910{,}000$ pound-inches. Total required $I/e = 557$ inches³. After trying various combinations of beams, the results are as follows, and No. 1 is adopted:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	3	186.0	24	90	185.8	7.13	7.3
2	4	139.5	24	79.9	173.9	7.0	2.7

In designing the lower tier, letting x = the distance from the left end of grillage to the section in question, the expression for the moment of the upward forces is $\frac{4000x^2}{6}\left[3b - \frac{(b-c)x}{L}\right]$. Hence, the expressions for bending moments under column 1, between the two columns and under column 2, are respectively as follows:

$$M(\text{column } 1) = \frac{4000}{2}\cdot\frac{x^2}{3}\cdot(49.8 - 0.471x) - \frac{500{,}000}{3}\cdot\frac{(x - 2.125)^2}{2}$$

$$M\ (\text{between columns}) = \frac{4000}{2}\cdot\frac{x^2}{3}\cdot(49.8 - 0.471x) - 500{,}000(x - 3.625).$$

$$M(\text{column } 2) = \frac{4000}{2}\cdot\frac{x^2}{3}\cdot(49.8 - 0.471x) -$$

$$500{,}000(x - 3.625) - \frac{400{,}000}{2.75}\cdot\frac{(x - 12.25)^2}{2}.$$

To get the values of x for the maximum value of M in each of the above equations, equate dM/dx to zero, which gives 3.42, 8.58 and 13.91 feet, respectively. Substituting these three values of x in the preceding equations, the corresponding values of M are 236,000, 231,000 and 232,000 foot-pounds. The maximum maximorum is, therefore, 236,000 foot-pounds. Total required $I/e = 177$ inches³. Trying various combinations of beams, there is obtained:

No.	Number of beams	I/e required	Size of beam Inches	Pounds	I/e furnished	Width of flange, inches	Clearance, inches
1	10	17 7	9	21.8	18.9	4 33	5.9 to 17.3
2	12	14.7	8	20.5	15.1	4.08	4.3 to 13.6
3	14	12.7	8	18.4	14.2	4.00	3.1 to 10.9
4	16	11.0	7	17½	11.1	3.75	2.5 to 9 3

No. 4 will be adopted.

Analysis will show that both foregoing designs are safe in shear and buckling.

Reinforcing bars should be placed in the concrete near the upper surface for the wider half of the footing.

Where more than two columns have a common footing, the structure becomes statically indeterminate and unsolvable unless some assumption is made regarding the deflection of the beams at the column bases. On the assumption that the points of application of P, Q and R of Fig. 158*c* remain in a straight

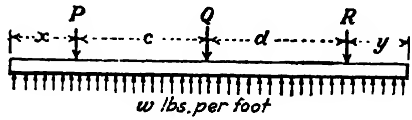


FIG. 158*c*.

line after deformation of the grillage, it can be proved by deflection formulas that

$$\frac{8c^2P - w(6x^2c^2 + 8xc^3 + 3c^4)}{8d^2R - w(6y^2d^2 + 8yd^3 + 3d^4)} = -\frac{c}{d}.$$

Applying the two laws $\Sigma Y = o$ and $\Sigma M = o$ to the structure taken as a free body, together with this equation, the values of x, y and w can be found.

As an example, let the loads P, Q and R be respectively, 600, 900 and 1500 tons, $c = 12$ feet and $d = 30$ feet. The values of x, y and w are found to be 7.8 feet, 15 feet and 46.3 tons, respectively. If the safe bearing capacity of the soil is 5 tons per square foot, the width of footing would be 9.5 feet.

ART. 159. DISTRIBUTION OF PRESSURE ON BASE

There is some question regarding the error involved in the assumption that the pressure from the footing is uniformly distributed on the ground. Taking the case of the single-column square footing, it is evident that the base of the footing will assume a saucer-like shape, and as a consequence the pressure will be a maximum at the center and a minimum at the outside. The law governing the variation of pressure will depend on the relative deflections of different points on the base of the footing, as well as on the modulus of compressibility of the soil and the thickness of the compressible stratum. Where the modulus is low and the thickness considerable, the slight difference in total deformation at different points will

cause but a slight difference in pressure. Where the soil is compressible but inelastic, or soft and subject to lateral flow, a fairly uniform distribution of pressure quickly obtains.

Where the material has a high modulus of compressibility, as in shale or rock, the footing should be designed for stiffness as well as for strength, or else the surface of the material should be shaped to fit the curve taken by the base of the footing when fully loaded, otherwise the pressure will be very unevenly distributed. For example, by using a stress-strain diagram of the values obtained in the foundation tests ot the St. Paul Building, New York City,[1] a theoretical solution shows that for the typical steel-grillage footing the pressure varies from a maximum at the center to approximately zero at the outside. The material on which the above foundation tests were made consisted of very compact sand, while the whole area of the lot was covered with a layer of concrete and steel beams buried in concrete, the tests being made through a hole.

ART. 160. STEEL GRILLAGE FOUNDATIONS

Most of the grillages used in the foundations for the Phelan Building, San Francisco, were 15 feet square, and made with

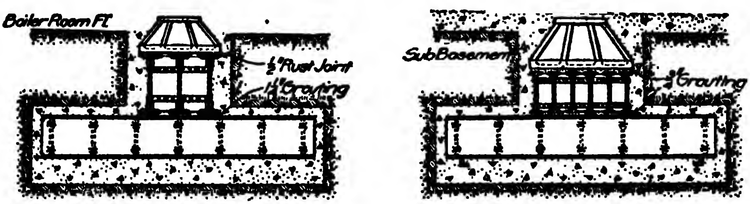


FIG. 160*a*.—Footings with Plate Girders and I-Beams in Double Tiers.

two cross-tiers of I-beams from 18 to 24 inches in depth, or with an upper tier of built-up girders and a lower tier of I-beams, as shown in Fig. 160*a*. The complete grillage plan is shown in Fig. 160*b*.

[2] "All footings are made with a bed of concrete 12 inches thick and 12 inches wider and longer than the dimensions of the

[1] See Engineering Record, vol. 33, page 388, May 2, 1896.

[2] Engineering Record, vol. 57, page 366, March 28, 1908.

first tier of grillage beams. In the upper part of the concrete there are two full-length rectangular grooves transverse to the lower tier of grillage beams. In each groove a 3-by 3-by 5/16-inch angle was carefully leveled with the upper edge of its vertical flange truly horizontal and 3/4 inch above the surface of the concrete. These serve as leveling bars to receive the lower flanges of the grillage beams and to insure their exact

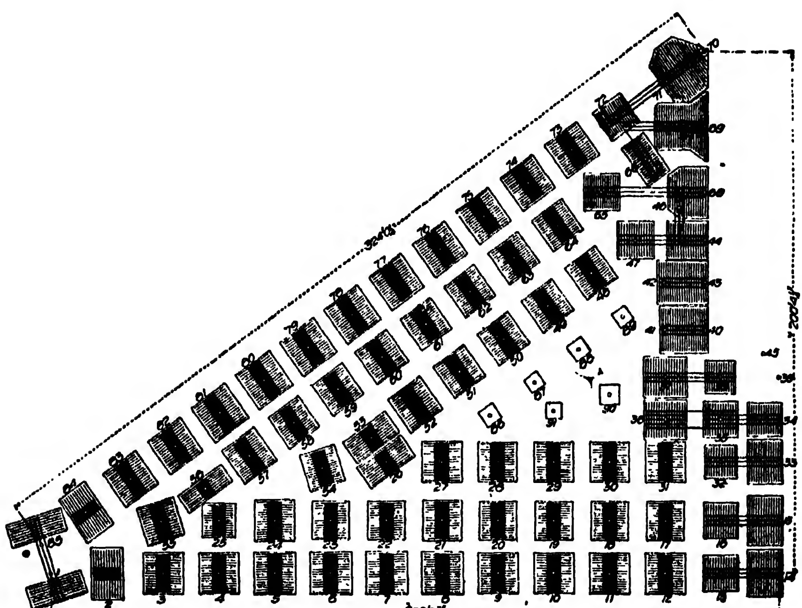


FIG. 160*b*.—Grillage Plan of Phelan Building, San Francisco, Cal.

height. The spaces between the beams and the concrete footings were grouted, the second tier of beams was shimmed 3/4 inch above the top flanges of the lower tier and grouted, the cast-iron pedestals were set 3/4 inch above the top flanges of the distributing beams and grouted, and a solid mass of concrete was filled in 6 inches around the outer edges of the beams and pedestals and up to the cellar floor, completely inclosing and protecting all the substructure steel work."

Figure 160c illustrates a very heavy grillage foundation for four columns of the Curtis Building, Philadelphia. It was necessary to use a single grillage for the four columns because of the short distances between the latter. The distributing girders for columns 254 and 255 have 48- by 1 1/16-inch webs

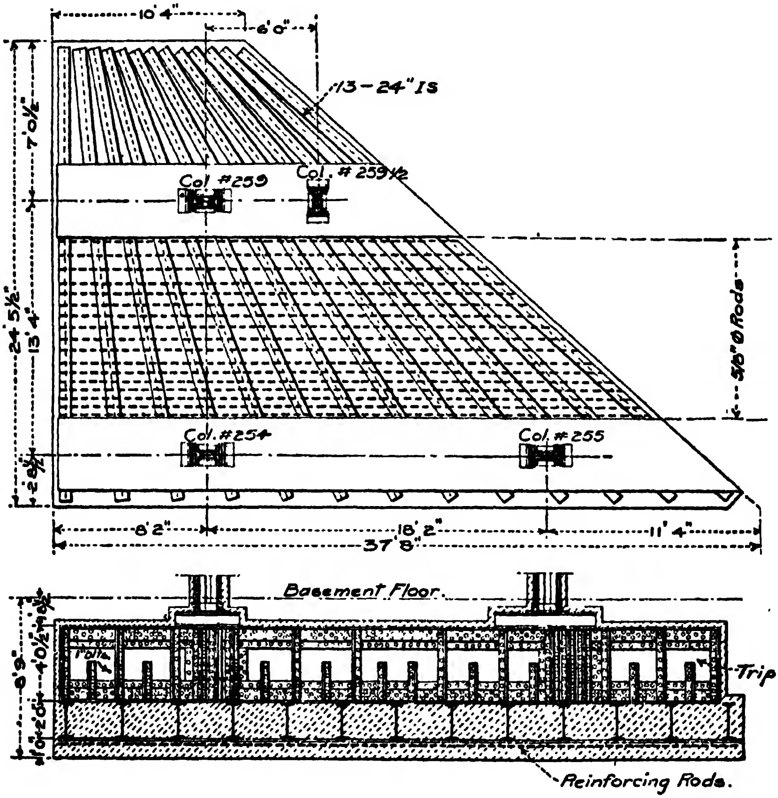


FIG. 160c.—Special Footing for Four Columns, Curtis Building, Philadelphia.

reinforced by 5- by 3- by 3/8-inch vertical stiffener angles and two 13- by ½-inch vertical side plates, and the top flanges of the girders are connected by transverse tie plates. The column loads are transmitted to the triple distributing girders by bolsters made of solid slabs of plain square steel billets which

are bolted to the upper flanges of the girders. The concrete footing is reinforced with rods for part of the base, due to the fact that the I-beams are there a considerable distance apart, thus developing beam action in the concrete.

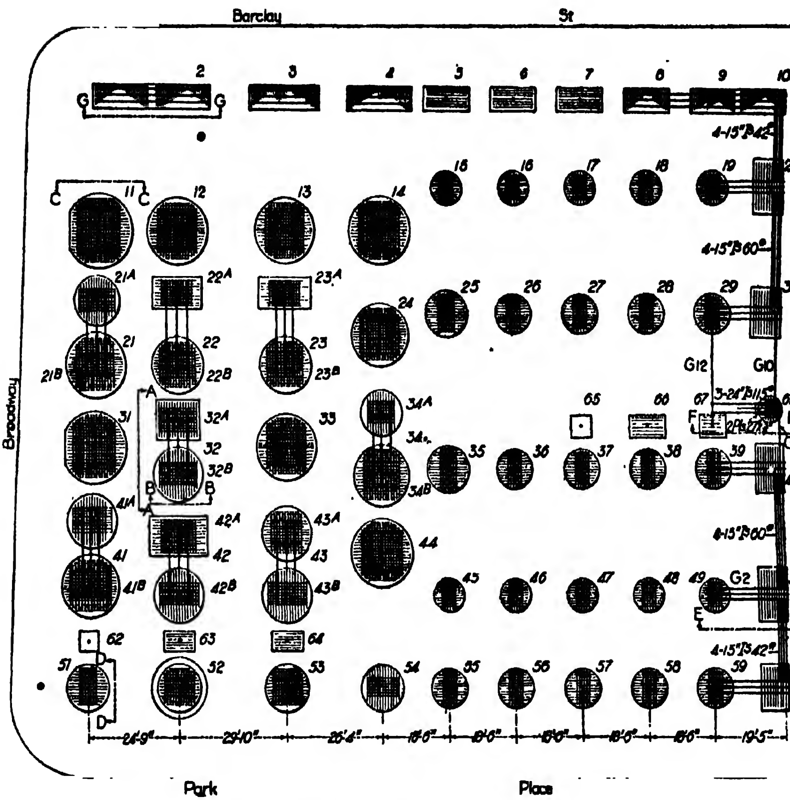


FIG. 160*d*.—Plan of Piers and Grillages for the Woolworth Building.

The Woolworth Bu lding, New York City, is founded on solid rock 115 feet below the curb level. The loads are carried from the columns to bed rock through grillage footings resting on reinforced-concrete piers. Figure 160*d* shows the general layout for the foundation, while Fig. 160*e* shows some of the details.

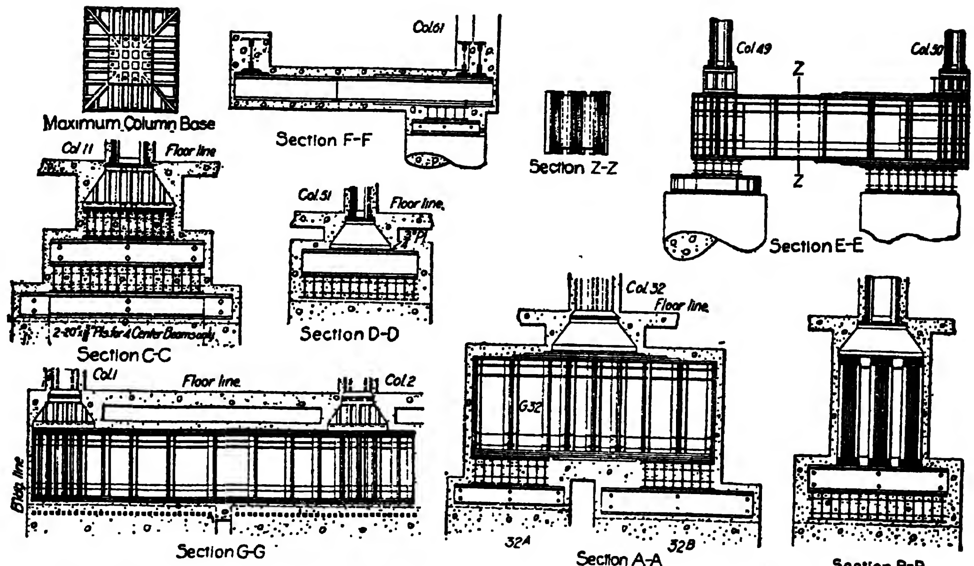


FIG. 160e.—Column Footings of Plate Girders and Grillages, Woolworth Building, New York City.
For column 11 the diameter of the pier is 18 feet 9 inches and the column load is 9,423,000 pounds. For this column the grillage is made of four tiers of 24 inch I-beams with flange reinforcement plates for the top tier and web reinforcement plates for the bottom tier. Section *E-E* shows the common method of supporting wall columns when the foundation must be kept back of the property line.

ART. 161. DESIGN OF REINFORCED-CONCRETE SPREAD FOUNDATIONS

Instead of serving merely as a protection for the steel, concrete may be made to take the load by using a reinforced-concrete footing in place of the I-beam grillage, thus lessening the cost of the foundation. Another advantage possessed by a reinforced-concrete foundation is that it can be cast in any shape or form desired. It may be in the form of a flat slab or of the slab-and-beam type (Fig. 163*a*). The former uses more concrete, while in the latter the form work is more expensive. For some interesting modifications of the elementary type the reader is referred to Art. 163.

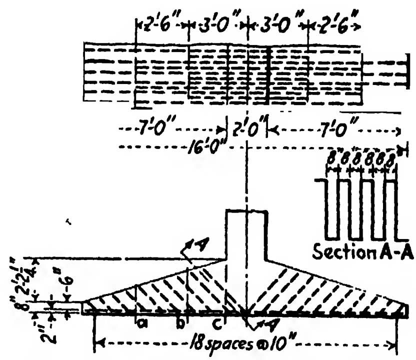


FIG. 161*a*.—Reinforced-Concrete Wall Footing.

DESIGN OF A REINFORCED-CONCRETE WALL FOOTING.—Assuming the load to be 64,000 pounds per linear foot of wall and the allowable bearing on the soil 4000 pounds per square foot, the width of footing will be 64,000/4000 = 16 feet. The thickness of the wall is 2 feet (Fig. 161*a*). The footing will be designed at three sections: at *a*, 5½ feet from the center of the wall; at *b*, 3 feet from the center; and at *c*, 1 foot from the center. Taking a 1-foot length of footing, the vertical shears and bending moments will be as follows:

$V_a = 4000 \times 2\frac{1}{2} = 10{,}000$ pounds. $M_a = 4000 \times 2.5^2 \times 12 =$ 150,000 pound-inches.

$V_b = 4000 \times 5 = 20{,}000$ pounds. $M_b = \frac{4000 \times 5 \times 12^2}{2} =$ 600,000 pound-inches.

$V_c = 4000 \times 7 = 28{,}000$ pounds. $M_c = \frac{4000 \times 7^2 \times 12}{2} =$ 1,176,000 pound-inches.

A 1-2-4 concrete will be used, with an allowable compressive unit-stress[1] in the concrete of $f_c = 600$ pounds per square inch and an allowable tensile unit-stress in the steel of $f_s = 16{,}000$ pounds per square inch. The ratio of the modulus of elasticity of steel to that of concrete will be assumed as $n = 15$. The depth to center of steel rods necessary to give a compressive stress in the concrete of 600 pounds per square inch is given by the formula $d = \sqrt{M/(Rb)}$, in which $R = f_c kj/2$. In the latter formula[2] $k = \sqrt{2pn + (pn)^2} - pn$ and $j = 1 - k/3$; $p = \frac{1}{2}\Big/\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)$. The work involved in getting the value of R will be greatly reduced by using the diagrams found in TURNEAURE and MAURER'S Reinforced-Concrete Construction. For the problem at hand the value of R is 95. Solving for d, $d_a = \sqrt{150{,}000/(95 \times 12)} = 11.5$ inches; $d_b = \sqrt{600{,}000/(95 \times 12)} = 23.0$ inches; and $d_c = \sqrt{1{,}176{,}000/(95 \times 12)} = 32.1$ inches ($32\frac{1}{4}$ inches being adopted). As it is inadvisable to use a depth at any section less than about 6 inches, the form shown in Fig. 161*a* will be adopted. The steel in the bottom will be given a 2-in. insulation.

The area of steel required at each section is given by the formula $A = M/(f_s jd)$. Using the values of d obtained above, so that the footing be equally strong in tension and compression:

$$A_a = 150{,}000/(16{,}000 \times 0.88 \times 11.5) = 0.92 \text{ square inch.}$$
$$A_b = 600{,}000/(16{,}000 \times 0.88 \times 23.0) = 1.85 \text{ square inches.}$$
$$A_c = 1{,}176{,}000/(16{,}000 \times 0.88 \times 32.1) = 2.60 \text{ square inches.}$$

Using a rod spacing of 3 inches center to center, there will be 4 rods in 1 foot of length of the footing. The required

[1] In a wedge-shaped beam the greater principal stress at the outer fibers act parallel to the upper surface of the beam and with an intensity equal to the maximum normal stress on a vertical plane divided by $\cos^2 \alpha$, in which α is the angle of inclination of the upper surface of the beam; hence, the allowable bending unit stress should be taken equal to the safe compressive stress in the concrete multiplied by $\cos^2 \alpha$.

[2] Based upon the assumption that the normal stress in the concrete on any vertical section varies as a straight line and that the stress in the steel equals n times the stress in the concrete. For formulas based on a different assumption, see Proceedings of the American Society of Civil Engineers, vol. 39, page 2067, November, 1913.

area of each rod will be $2.60/4 = 0.650$ square inches. A 1-inch round deformed rod, giving an area of 0.785 square inch, will be adopted. Three rods will furnish the required area at b, while two rods will furnish that required at a; hence, certain of the rods may be bent up or cut off as shown in Fig. 161*a*.

Using an allowable bond unit-stress of 125 pounds per square inch of rod surface, the necessary length of rod to develop full strength is $(16{,}000 \times 0.785)/(125 \times 3.14) = 32$ inches. Computing the bond stress in the rods by the formula[1] $u = (Vd - M \tan \alpha)/(jd^2\ \Sigma o)$, in which $\tan \alpha$ is the slope of the upper surface of the footing and Σo the perimeter of the rods at the section in question, the values are as follows:

$$u_a = (10{,}000 \times 15.4 - 150{,}000 \times 0.312)/(0.88 \times 15.4^2 \times 6.28) = 82 \text{ pounds per square inch.}$$

$$u_b = (20{,}000 \times 24.75 - 600{,}000 \times 0.312)/(0.88 \times 24.75^2 \times 9.42) = 61 \text{ pounds per square inch.}$$

$$u_c = (28{,}000 \times 32.25 - 1{,}176{,}000 \times 0.312)/(0.88 \times 32.25^2 \times 12.56) = 47 \text{ pounds per square inch.}$$

All of these values are well below the safe limit of 125 pounds per square inch.

Assuming that the concrete takes no longitudinal tension, the maximum intensity of diagonal tension is given by the formula $t = (Vd - M \tan \alpha)/(jd^2)$. A shorter method of computing the maximum diagonal tension is by taking the bond stress values and multiplying them by the perimeter of the rods. Thus,

$$t_a = (82 \times 6.28)/12 = 43 \text{ pounds per square inch.}$$
$$t_b = (61 \times 9.42)/12 = 48 \text{ pounds per square inch.}$$
$$t_c = (47 \times 12.56)/12 = 49 \text{ pounds per square inch.}$$

Although conservative specifications limit the allowable diagonal tension to 40 pounds per square inch, the above can be safely carried by the concrete without reinforcement, but to illustrate the method stirrups will be designed to carry all of this tension. Placing the stirrups on a 45-degree slope and

[1] Only approximately true when p is not constant.

using ½-inch round deformed rods with two prongs in a 16-inch length, as shown in Fig. 161*a*, the strength of one line of stirrups in a 12-inch length will be $16{,}000 \times .196 \times {}^{12}\!/_{8} = 4700$ pounds. Denoting the horizontal distance between rows of stirrups by s, the formula is $s = 4700/(12 \times t \times \cos 45°)$, giving

$$S_a = 4700/(12 \times 43 \times 0.707) = 12.5 \text{ inches.}$$
$$S_b = 4700/(12 \times 48 \times 0.707) = 11.5 \text{ inches.}$$
$$S_c = 4700/(12 \times 49 \times 0.707) = 10.5 \text{ inches.}$$

A uniform spacing of 10 inches will be adopted.

In this type of beam the maximum intensity of vertical shear occurs at the top and equals $f_c \tan \alpha$, where α is the inclination of the upper surface of the slab. The shearing stress is therefore $600 \times 0.312 = 187$ pounds per square inch.

ART. 162. DESIGN OF REINFORCED-CONCRETE COLUMN FOOTINGS

The stresses in a reinforced-concrete footing for a column are due more to flat-slab action than to beam action and hence are much less determinate than in the wall footing. However, the stresses may be approximately analyzed by either flat-slab or beam formulas. The former method is not entirely satisfactory, due partly to the necessary approximations of any formulas based on the theory of the flat plate, and partly to the tedious computations involved unless specially prepared tables or diagrams are used. For an example of the design of a footing based on the flat-slab principle, see TAYLOR and THOMPSON'S Concrete, Plain and Reinforced.

Where beam formulas are used, it is generally assumed that the section of maximum bending moment and shear is at the outer face of the column. If the footing has a two-way reinforcement the stress cannot be uniformly distributed over this section. For instance, looking at Fig. 162*a*, the load from the soil at point *c* will evidently go to the column through *dc* acting as a cantilever beam. On the other hand, a part of the

load at *a* will first go to some point, as *c*, through *ac* acting as a beam, and the balance to some point, as *b*, through *ab* acting as a beam. The part which goes to *c* will then go to *d* through *cd* acting as a beam, while the part which goes to *b* will go to *e* through *be* acting as a beam. Thus, it is evident that the stress along the plane *A–A* will vary from a maximum at the column face to a minimum near the sides of the footing.

From experiments made in the testing laboratory at the University of Illinois, A. N. TALBOT summarizes the proper method of design as follows: [1]"For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point halfway out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section . . .

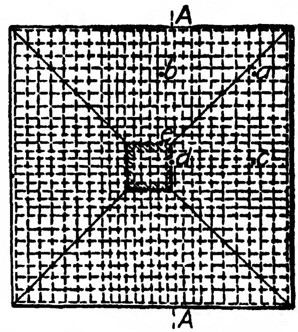


FIG. 162*a*.—Column Footing of Reinforced Concrete.

"With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edges of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement spaced uniformly over the footing, the method proposed, for determining the maximum tensile stress in the reinforcing bars, is to use in the cal-

[1] Bulletin 67, Engineering Experiment Station, University of Illinois.

culation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stresses."

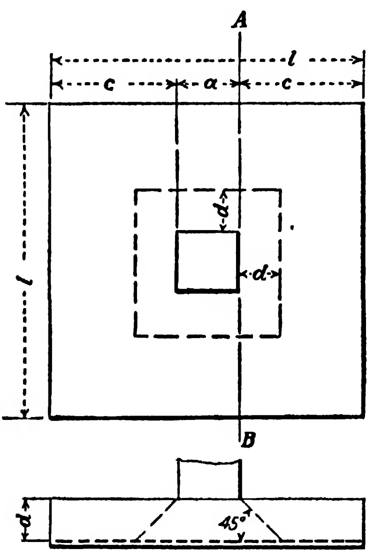


FIG. 162*b*.

The formula for moment is, then (Fig. 162*b*), $M = w(0.5ac^2 + 0.6c^2)$ and the effective width over which the moment is resisted is $b = a + c + d$, where w is the intensity of soil pressure upward. The depth of slab and the amount of steel in the width b are given by the formulas $d = \sqrt{M/Rb}$ and $A = M/(f_s jd)$, as given in Art. 161.

"The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond-stress formula, and to consider the circumferences of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section."

The formula for this shear is $w(ac + c^2)$ and the unit bond is $w(ac + c^2)/\Sigma ojd$, where Σo is the perimeter of the bars in the distance b.

To get the critical section for diagonal tension, TALBOT recommends taking a vertical section formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing. The total shear is $w[l^2 - (a + 2d)^2]$ and the intensity of diagonal tension is $w[l^2 - (a + 2d)^2] / 4(a + 2d)jd$.

Punching shear should be calculated for the vertical sections which inclose the pier footing. The total punching shear is $w(l^2 - a^2)$ and the maximum intensity is $w(l^2 - a^2)/4ajd$.

The foregoing is based on a slab having a horizontal upper surface.

DESIGN OF A FOUR-WAY REINFORCED FOOTING.—A footing with four-way reinforcement (Fig. 162c) is more susceptible of a rational analysis than the two-way reinforced footing. Tests by A. N. TALBOT (see previous reference) show that this type gives a somewhat stronger footing than the two-way type.

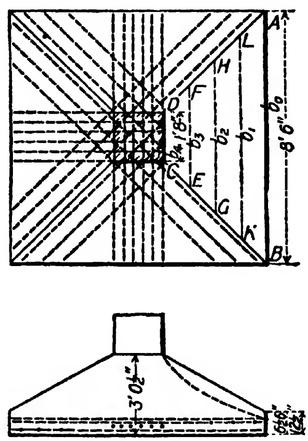


FIG. 162c.—Reinforcement for Column Footing.

Assuming the load to be 210,000 pounds and the allowable bearing on the soil 3000 pounds per square foot, the area of the footing will be 210,000/3000 = 70 square feet. A base 8 feet 6 inches square will be used. The column base will be assumed as 20 inches square.

In this design the part $ABCD$ in Fig. 162b will be assumed to act as a free cantilever about CD, as will also $ABEF$, $ABGH$ and $ABKL$ about EF, GH and KL, respectively; in other words, it will be assumed that there is no stress on the planes AD and BC. Dividing the horizontal distance between AB and DC into four equal parts by the lines b_1, b_2 and b_3, the lengths of the lines b_0, b_1, b_2, b_3 and b_4 are, respectively, 8.50, 6.79, 5.08,

3.37 and 1.67 feet. Let A_1, A_2, A_3 and A_4 represent, respectively, the areas of the base of the footing to the right of the b lines of the corresponding subscripts, then their values will be $A_1 = 6.54$, $A_2 = 11.6$, $A_3 = 15.2$ and $A_4 = 17.35$, all expressed in square feet.

The upward pressure from the soil is $210{,}000/(8.5)^2 = 2910$ pounds per square foot. The shears on the sections b_1, b_2, b_3 and b_4 are, respectively, 19,000, 33,800, 44,200 and 50,500 pounds. The moment of the upward pressure to the right of and about b_1 is $19{,}000 \times \frac{2 \times 8.5 + 6.79}{8.5 + 6.79} \times \frac{0.854}{3} \times 12 =$ 101,000 pound-inches. The moments of the forces to the right of and about b_2, b_3 and b_4 are, respectively, 376,000, 775,000 and 1,267,000 pound-inches. Using an allowable unit stress for the rods of 16,000 pounds per square inch and for the concrete of $650 \cos^2 \alpha = 500$ (approximately) pounds per square inch, in which α is the angle made by the upper surface with the horizontal, the values of d as given in the formula $d = \sqrt{M/(Rb)}$ are $d_1 = 4.2$, $d_2 = 9.3$, $d_3 = 16.4$ and $d_4 = 29.8$ inches.

Using the formula $A = M/(f_s jd)$ to get the required area of cross-section of steel at b_1, b_2, b_3 and b_4, the respective values are 1.69, 2.83, 3.30 and 2.97 square inches. Assuming 12 deformed rods, the required area of each one is $3.30/12 = 0.275$ square inches. A 5/8-inch round rod furnishes an area of 0.307 square inch. The rods will be placed as shown in Fig. 158*b*, each layer being 1½ inch above the one below it.

The ordinates to the curved line in Fig. 158*b* represent the required depths, but, as shown in the same illustration, the depths adopted will be greater than these.

The bond stresses as given by the formula $u = (Vd - M \tan \alpha)/(jd^2 \Sigma o)$ are 55, 57, 50 and 41 pounds per square inch for the sections b_1, b_2, b_3 and b_4, respectively.

The maximum unit shear is $f_c \tan \alpha = 500 \times 0.586 = 293$ pounds per square inch. This is a rather high value but, as it occurs at the point of maximum compression and so does not develop a heavy diagonal tension, it may be considered safe.

Assuming that the concrete takes no direct tension, the maximum diagonal tension for each section, as given by the formula $t = (Vd - M \tan \alpha)/(bjd^2)$, is $t_1 = 16$, $t_2 = 22$, $t_3 = 29$ and $t_4 = 48$ pounds per square inch. Hence, stirrups are required for only a short distance from the face of the column. The method of design of the same is treated in Art. 161 and will not be repeated here.

The design of the slab-and-beam type of footing follows closely the method of design of slabs and beams in building construction. The slab serves as a beam to carry the load from the soil to the beam, the span being taken as the distance center to center of beams; and the latter, acting as cantilevers, carry it to the column. Where the beams have constant cross-sections the formulas for stresses as derived in any standard treatise on reinforced concrete are applicable, and where tapered, the formulas given in Art. 161 may be used.

Where one footing serves for two columns, the method of obtaining the shape of footing, as well as the shears and bending moments, is similar to that for the I-beam grillage (Art. 158), while the standard formulas are applicable in finding the stresses. On page 647 of the second edition of TAYLOR and THOMPSON'S Concrete, Plain and Reinforced, an example of this type of footing is worked out.

ART. 163. CONCRETE SPREAD FOUNDATIONS

Two standard forms of the reinforced-concrete spread footings, used for the column foundations of a railway terminal station at Atlanta, Ga., are shown in Fig. 163*a*. The one illustrated on the left was used for 20- by 24-inch columns and was in the form of a truncated pyramidal slab reinforced with bars and stirrups. The one shown on the right was of the beam-and-slab type. The details are sufficiently shown to require no explanation.

The 125-foot concrete block chimney for the St. Joseph's Home, Chicago, was founded on a blue clay, the base of the foundation extending about 5 feet below the surface of the

ground. The footing, shown in Figs. 163*b* and *c*, consists of a circular slab 24 feet in diameter and 10 inches thick, on which is built a box with a square outer surface 8 feet 3 inches on a side, and with an octagonal inner surface about 6 feet 7 inches between opposite faces. The box is about 4 feet high.

[1] "From either corner of this box extends a series of eight cantilever ribs reaching approximately to the outer edge of the

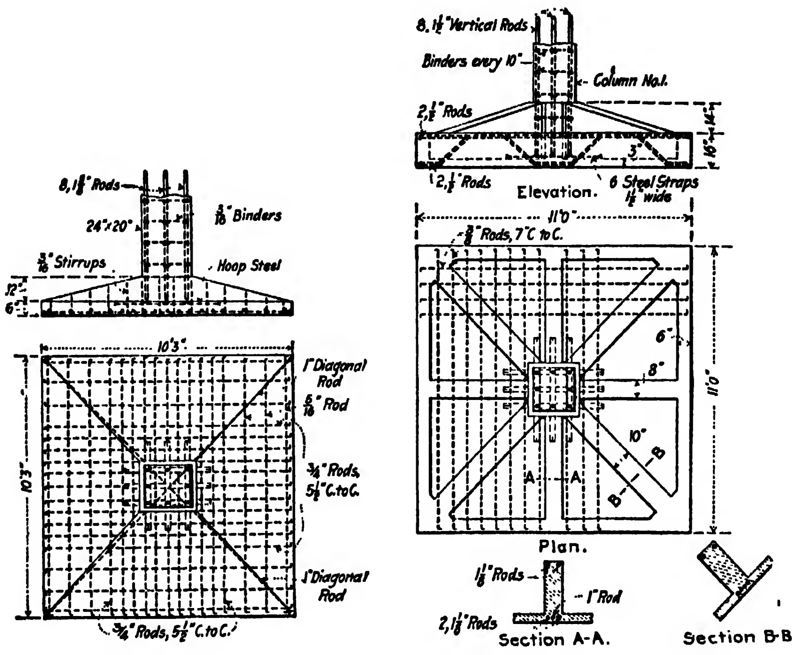


FIG. 163*a*.—Pyramidal and Ribbed Slab Footings of Reinforced Concrete, Atlanta Terminal Station, Southern Railway.

slab as shown in the accompanying view. These cantilever ribs are each 14 inches wide, 4 feet deep at the box and slope off uniformly to a width of 8 inches at the top of the slab. Their effective depth is virtually 4 feet plus the effective depth of the slab, as they are built integral with it; and their reinforcement, which consists of 1-inch round bars and ¼-inch vertical stirrups, extends up from the lower surface of the slab, as

[1] Engineering Record, vol. 65, page 636, June 8, 1912.

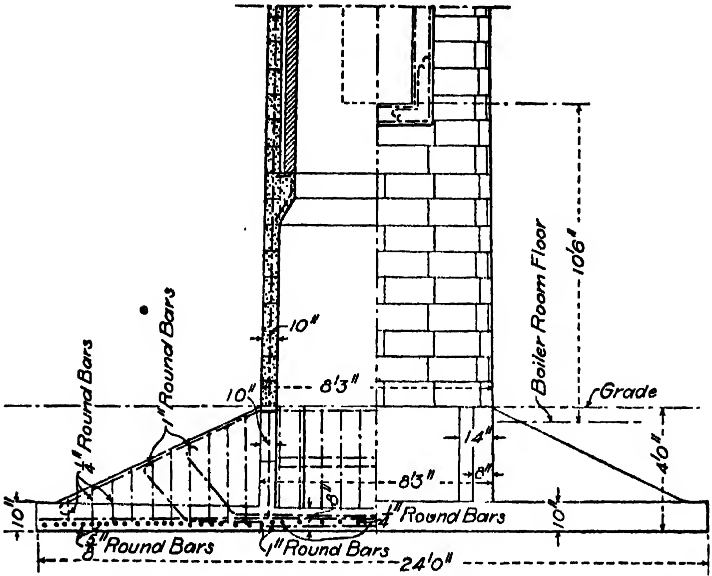


FIG. 163*b*.—Slab and Box Footing of Reinforced Concrete for a 125-Foot Chimney in Chicago.

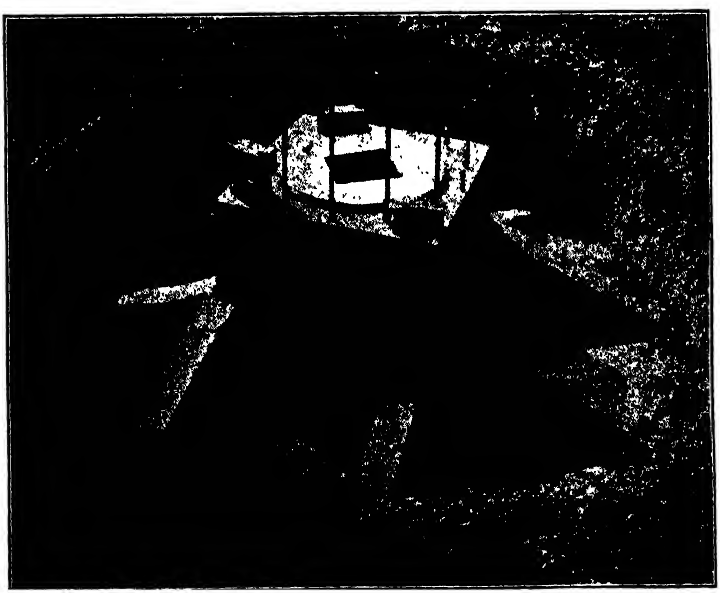


FIG. 163*c*.—View of the Same Footing as Shown in Fig. 163*b*.

shown in the accompanying drawing. The base is thus made up of a series of slabs, each supported by the adjacent cantilever ribs and reinforced with ⅝-inch round bars spaced according to the position of the slab in the base. That portion of the base inclosed at the center is reinforced with a double system of ¼-inch round bars, spaced 6½ inches on centers."

Figure 163*d* represents a novel type of foundation used for a loft building in New York City. There were three lines of columns, two lines of wall columns and one line through the center. The foundation presented something of a problem because the adjoining structure rested on a pile foundation, which the architect feared was in a poor condition. On account of the desire not to be forced to the expense of underpinning this adjoining building, a deep foundation was out of the question. The simple spread footing could not be used for the wall columns because of lack of space. As finally constructed, the foundation consisted of a solid framework of reinforced-concrete beams.

[1]"The special feature of the cantilever construction is that the one cross-beam and a portion of each longitudinal beam form a T-section, the center of gravity of which is the same as the center of gravity of the column loads plus the weight of the side wall. Thus, looking at Fig. 163*d*, it will be seen that half of the load coming on the column in the center of the building and the whole load coming on a wall column and the wall load adjacent to that column is carried on that portion of the side concrete beam and the cross-beam there shown, and that the center of gravity of these loads is the same as the center of gravity of the T-beam formed by the side beam with the transverse beam going at right angles from it. The variation in the loads and, consequently, in the centers of gravity, resulted in different shapes and sizes of the supporting beams."

For the foundations of a 12-story concrete building, 108 by 162 feet in plan, a slab under the entire building, 5 feet thick under the outside walls and 4 feet thick in the interior, with four-way reinforcement, both top and bottom, was used. The

[1] Engineering News, vol. 68, page 995, Nov. 28, 1912.

column spacing was approximately 20 by 26 feet, the columns resting on stepped pedestals 12 feet square by 2 feet thick and 5½ feet square by 2 feet thick.

In the construction of a building on the tidal marsh of Newark Bay,* where the soil was soft, peaty, alluvial marsh ground to a depth of about 35 feet, piles were the logical solution, but the loading was so heavy that piles over the whole area, at 2½-foot

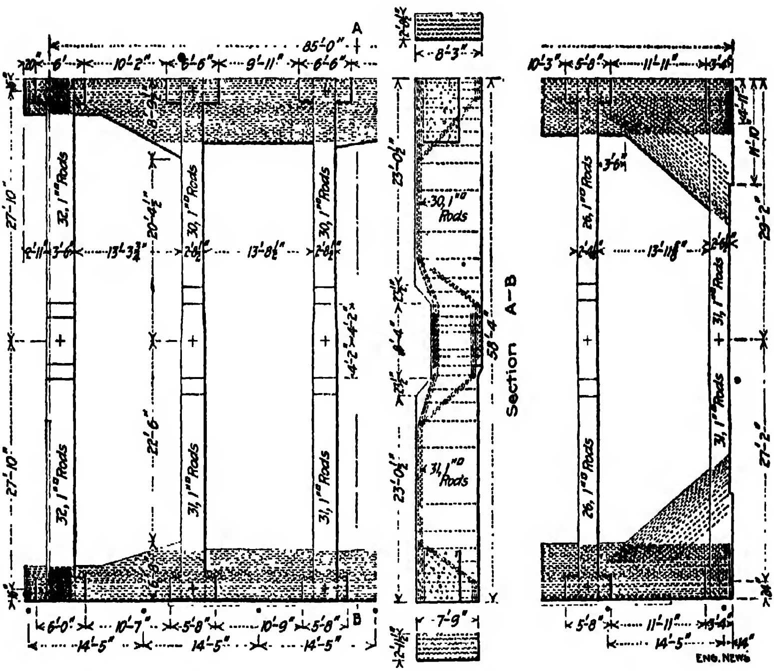


FIG. 163*d*.—Plan of Reinforced-Concrete Cantilever Footings of 12-Story Loft Building, 25–29 West 31st Street, New York City.

spacing, were necessary. The ground was excavated to below low water, after which the piles were driven and the space between the piles refilled with cinders and earth to within 9 inches of the pile tops. An unreinforced-concrete mat was then placed over the piles, this mat being 3 feet thick for a part of the foundation and 5 feet for the remainder.

Reinforced-concrete spread foundations covering the whole area of the basement were used for the factories of Herman

Behr & Co., and W. H. Sweeney Mfg. Co., Brooklyn, N. Y. Figure 163*e* shows the details for the first-named factory. This raft foundation, which was of the beam-and-slab type, had a slab thickness of 1 foot and a beam thickness of 3 feet. The beams formed continuous lines under the outer wall and along the center line of the columns lengthwise of the building, the column spacing being 16 feet 10 inches longitudinally and approximately 19½ feet transversely. These beams were 5 feet wide under the walls and 6 feet wide under the columns. The intervening space between beams was brought up nearly to surface level by a dirt fill, and a finished concrete floor was

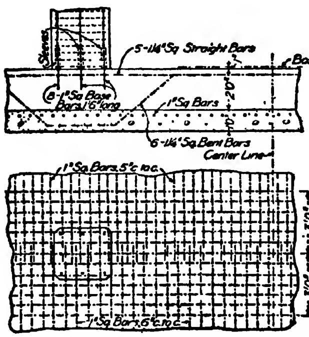


FIG. 163*e*.—Spread Foundation.

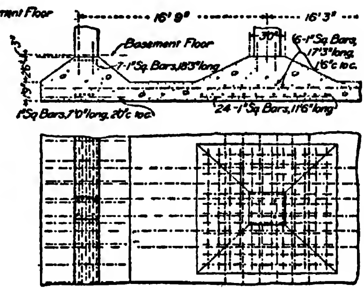


FIG. 163*f*.—Spread Foundation.

laid over the whole area. As shown in the illustration, the reinforcement for the 12-inch slab consisted of transverse bars 1 inch square, spaced 5 inches on centers and 3 inches from the top of the slab. The beams under the columns were reinforced with 11 1¼-inch square bars near the upper surface, the five center bars being carried through straight and the six outside bars bent down under the column.

The foundation of the W. H. Sweeney Mfg. Co.'s factory consisted of a slab over the whole area surmounted by truncated pyramidal slabs under all the columns and a trapezoidal-shaped slab under the wall, as shown in Fig. 163*f*. The columns were spaced approximately 16 feet on centers in both directions.

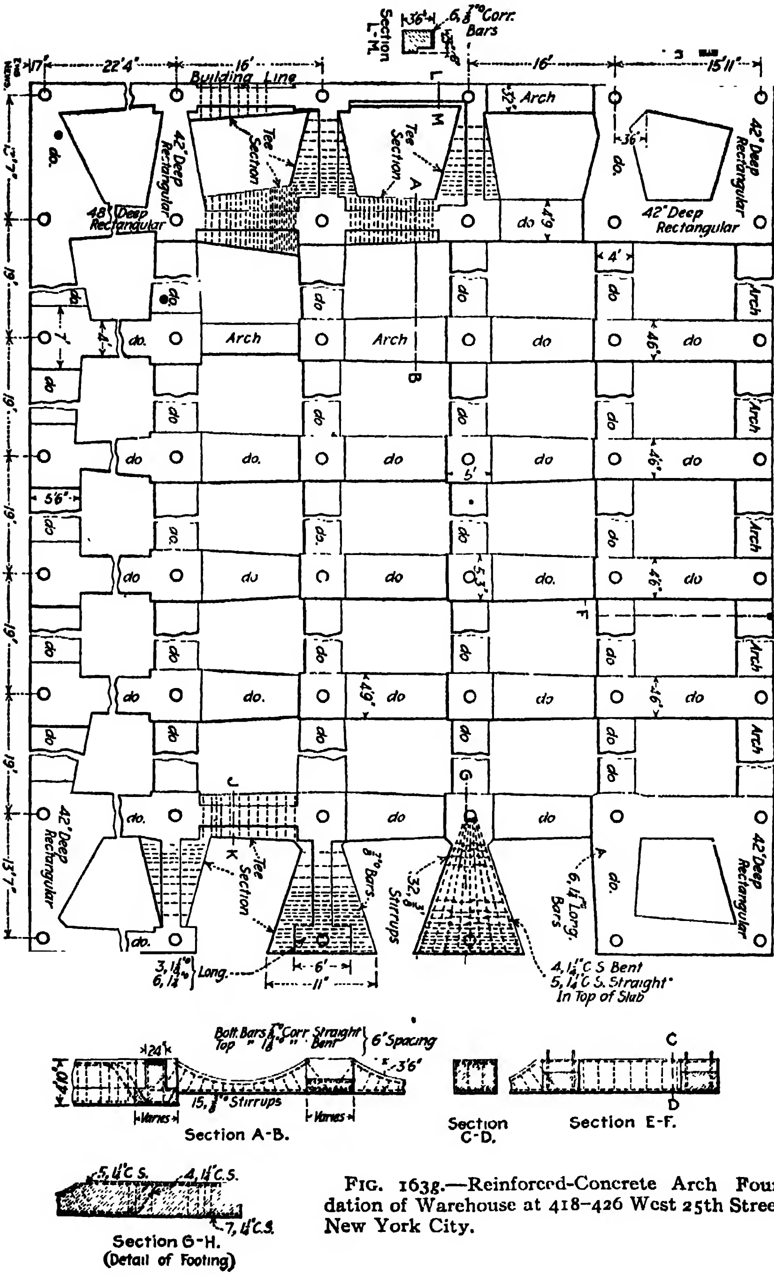


FIG. 163g.—Reinforced-Concrete Arch Foundation of Warehouse at 418–426 West 25th Street, New York City.

The column footings were raised 2 feet 6 inches above the top of the raft slab and the latter was reinforced with six lines of rods about 1½ feet on centers, and laid in both directions along the center lines of the columns. Further reinforcement was used in the bottom of the slab under the columns and walls, as shown in the illustration.

The inverted-arch foundation of reinforced concrete as used for a building in New York City presents an unusual type of spread foundation. Its adoption was due to the necessity of having a very shallow foundation. The limit of depth fixed by the architect was not sufficient for isolated reinforced-concrete footings and as steel I-beam grillages would have cost

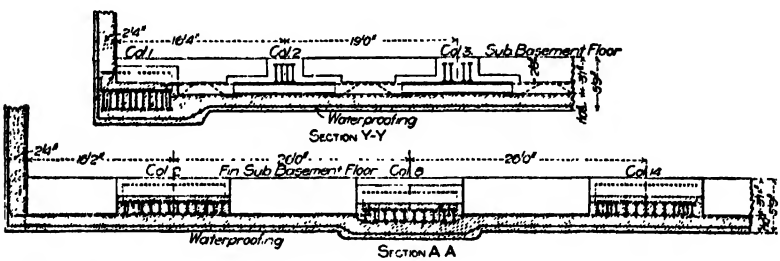


FIG. 163*h*.—Cellar-Floor Sections Showing Grillage Beams and Reinforced-Concrete Girders, Pope Building, Cleveland, Ohio.

about 25 percent more, the inverted-arch form was used. The arches ran in both directions between columns, as shown in Fig. 163*g*. They were 12 inches deep at the crown and 42 inches deep under the cast-iron column bases, and varied from 4 to 5 feet in width. The reinforcement consisted of ⅞-inch round, straight, corrugated bars in the bottom, spaced 6 inches on centers, and 1⅛-inch bent bars in the top, spaced the same distance. All end spans were made of rectangular or T-shaped concrete beams, to provide for the thrust in the adjoining arches.[1]

In the foundation for the Pope Building, Cleveland, Ohio, a combination of a steel grillage and a reinforced-concrete raft foundation was used. The material upon which the founda-

[1] For further details, see Engineering News, vol. 66, page 763, Dec. 28, 1911.

tion was placed consisted of a few feet of quicksand overlying clay. As the sides of the lot were inclosed by a permanent steel cofferdam extending well down into the clay, the quicksand was not subject to outside disturbance, and hence made a satisfactory cushion. A 6-inch layer of concrete was first spread over the bottom and covered with tar and felt waterproofing, after which a 16-inch layer of concrete was placed on the waterproofing. On this were located the I-beam grillages, as shown in Fig. 163*h*, section *A-A* being taken at right angles to the street and section *Y-Y* parallel with the street. The grillages were made of two tiers of 24-inch I-beams, each supporting a single column. In all the intermediate spaces the concrete floor slab was reinforced with rods, thus providing for the distribution of the column loads over the entire bottom.

CHAPTER XVI

UNDERPINNING BUILDINGS

ART. 164. NEEDLE-BEAM UNDERPINNING

The technical term "underpinning" is used to denote the placing of new foundations or supports under existing structures. As an engineering art and science this work has been developed almost entirely in a few large cities, notably New York, Chicago and Boston. In New York, the subways and the modern "sky-scraper," with its foundations carried far below those of surrounding structures, have compelled the placing of new and deeper foundations for many buildings. Some of these underpinned buildings have wall loads as high as 45 tons per linear foot and column loads of 300 tons or more. The underpinning of such heavy buildings requires great skill and care, for it must be done in such a manner that no settlement occurs; with the mechanical equipment of the modern office building, such as elevators, motors, engines, etc., a very slight differential settlement often causes trouble. Moreover, the work must often be done hastily and in a limited space.

The two general methods of underpinning are: first, by needle-beams to support the structure temporarily, after which the old foundations are removed and new ones placed; and, second, by vertical cylinders (without temporarily supporting the structure) in the plane of and under the walls, carried down to solid bearing.

The needle-beam method of underpinning may be called the indirect method, since the function of the needle-beams is merely to take the loads temporarily from the old foundation to permit removing the latter and the building of new foundations. This method is the older and more widely used, being universally applied where the new foundation is of a simple type and not carried to a great depth.

The fundamental principle of the needle-beam method consists in cutting holes through the walls of the building at intervals of from 3 to 10 feet or more, depending somewhat on the strength of the walls, and placing wooden or steel beams through

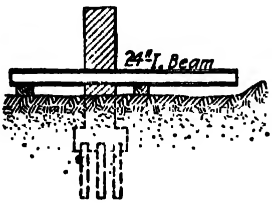


FIG. 164*a*.—First Step.

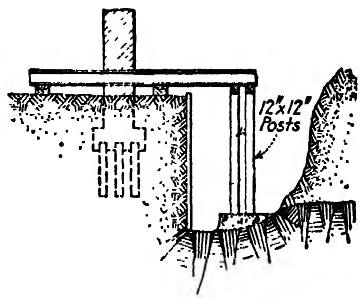


FIG. 164*b*.—Second Step.

these openings. The ends of the beams are held on temporary supports placed at a sufficient distance from the wall to permit excavation and reconstruction work to be carried on under the wall. The needles are raised by placing jacks under the ends

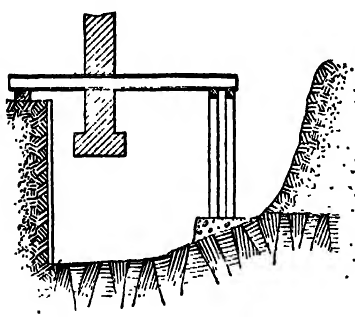


FIG. 164*c*.—Third Step.

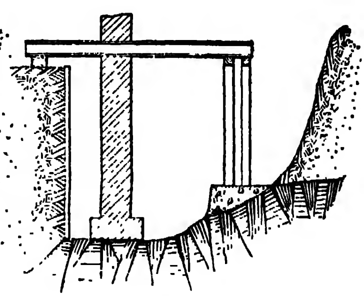


FIG. 164*d*.—Fourth Step.

of the beams until they take bearing on the wall and thus lift the latter from its old foundation.

Figures 164*a*–*d* illustrate the general method used in underpinning the Cross Building, New York City. [1] "The first step was to cut through the old brick wall, which was 56 inches thick,

[1] Engineering News, vol. 68, page 1134, Dec. 19, 1912.

an opening large enough to allow the entering of the needle-beams, made up of four 24-inch I-beams . . . The needles, which were spaced about 6 feet apart along the wall, were supported on the inside of the old building by blocks placed on the concrete cellar floor, and on the outside by blocks supported on the earth immediately alongside the wall. Sheathing was then driven outside of the blocks, and an excavation made to solid rock. On this rock a rough concrete footing was placed and 12- by 12-inch posts erected to carry the outside end of the needle-beams, the needle-beams then being supported on the inside by blocking on the concrete pavement and on the outside by heavy posts on a solid concrete footing. Shims were driven in under the brick wall for support and sheathing driven on the inside of the old building, as shown in Fig. 164*c*. Excavation was then made under the brick walls to rock bottom, and the entire old footing removed. A new concrete footing was placed on this rock bottom . . . "

Oftentimes conditions make it impossible to occupy the space on both sides of the wall, the space on the inside being perhaps occupied by a store or storage room; or the space on the outside is taken up with other construction work. In either case the method just described must be modified. One way of avoiding interior work is to use the figure-4 needle-beam as described in Art. 168. A number of arrangements may be employed to avoid occupying space outside the wall, among which the most widely used is the cantilever needling plan described in Art. 167; another scheme uses needle-beams of the regular type at considerable distances apart, the intermediate needles having their outside ends bearing on a truss or girder, parallel and close to the wall on the outside, the ends of the truss or girder bearing on the regular needles. This method materially reduces the space used on the outside.

DESIGN OF NEEDLE-BEAMS.—Probably the most difficult feature in the design of a needle-beam system lies in estimating the load on any particular member. The rest of the design is a matter of elementary mechanics and needs no discussion here. The total weight of the structure to be supported can usually

be approximated with sufficient accuracy; if the needle-beams are spaced at equal distances apart, it will ordinarily be assumed that all take the same load. To make this a fact, care should be exercised to have all jackscrews raised the same amount. A good scheme is to have one or two men do all this work, giving each jackscrew perhaps half a turn at a time.

Art. 165. Examples with Needle-Beams

Needle-beams are usually supported in one of the following ways: first, by struts resting on concrete bases; second, on piles; or, third, on cribbing built on the surface of the ground.

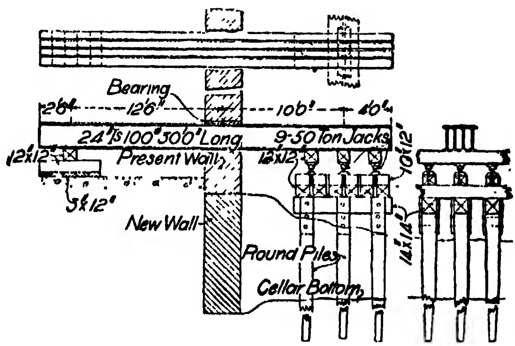


Fig. 165*a*.—Underpinning with Needle-Beams and Pile Bents.

The first method is satisfactory where the loads are not unduly large and where good bearing can be obtained; it also takes up the least space. Where the ground is soft, a pile foundation is the only satisfactory method of insuring absolute stability. The crib form may be used where the loads are large and must be distributed over a considerable area of the ground. Figure 164*d* illustrates the strut method of support, the details of which are described in Art. 164.

Figure 165*a* shows the details of the method used for underpinning buildings adjacent to the Adams Express Building, New York City. Here the inside ends of the needles were supported on blocking resting on the cellar floor, while the

outside ends rested on 12- by 12-inch timbers running parallel to the wall, under which were the 50-ton jacks used in raising and supporting the wall. These in turn rested on small blocks which took bearing on longitudinal 12- by 12-inch timbeıs, the latter resting on pile bents.

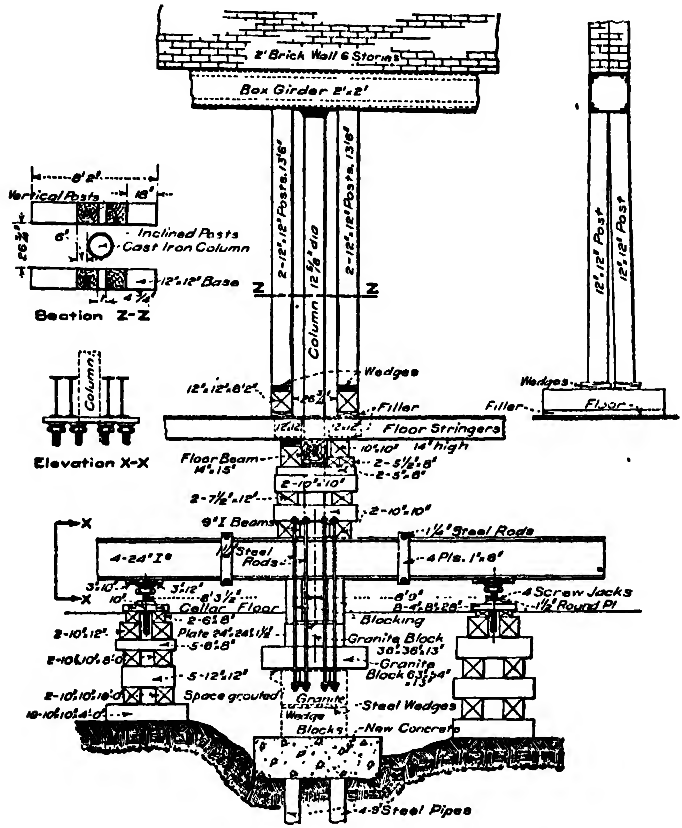


FIG. 165*b*.—Underpinning a 300-Ton Column on Quicksand, Sargent Building, New York City.

Figure 165*b* illustrates the form of needling which uses only cribwork for its support. The needle-beams, of which there are four, support a 300-ton column. [1] "The first-floor beams were blocked and wedged up on the girders close to the columns

[1] Engineering Record, vol. 61, page 649, May 14, 1910.

and sills were laid across them on the first floor adjacent to the column to receive two pairs of posts wedged to bearing on the under side of the box girder close to the column. The wedges were driven and the jacks operated to take the floor and wall loads from the column to the cribbing and to compensate for any settlement of the latter."

Figure 166*a* shows the method of underpinning the Benedict Building, New York City. The needle-beams rested on struts on the outside and cribbing on the inside. Holes about 5 feet apart were first cut in the wall and into these holes were inserted needle-beams composed of 15-inch I-beams in groups of three, each group being tied together with iron yokes at both ends. On the outer end of the needle-beams two 20-ton jack-screws rested on two 12- by 12-inch vertical posts and took bearing against horizontal steel plates on the lower flanges of the I-beams. The posts took bearing at their lower ends on 5- by 5-foot grillages of 12- by 12-inch timbers resting on the concrete footing.

ART. 166. SUPPORTING WALL BELOW BEAMS

With the needle-beam method of underpinning it is usually impracticable to support the wall from below the old foundation. For this reason, if the new foundation is to be constructed only up to the old, it becomes necessary to use some special method of supporting the wall and old foundation below the needling.

In the case of the Benedict Building (Fig. 166*a*), this was done as follows: [1] "Narrow excavations were made between the old wall and the sheeted pits, and the latter were braced against the face of the masonry as the excavation proceeded. When it reached the bottom of the old footing, small drifts were extended under it and in them 'springing needles,' each consisting of a pair of 12- by 12-inch horizontal timbers bolted to the vertical shores, were inserted with their ends bearing against the bottom of the old footing. Vertical chains with turnbuckle adjustments were attached to the I-beam needles above, close

[1] Engineering Record, vol. 55, page 267, Mar. 2, 1907.

to the face of the wall, and engaged the springing needles, formed fulcrums for the latter which acted as cantilevers supporting the footing below the main needle-beams. A vertical strut was inserted between the ends of the springing beams

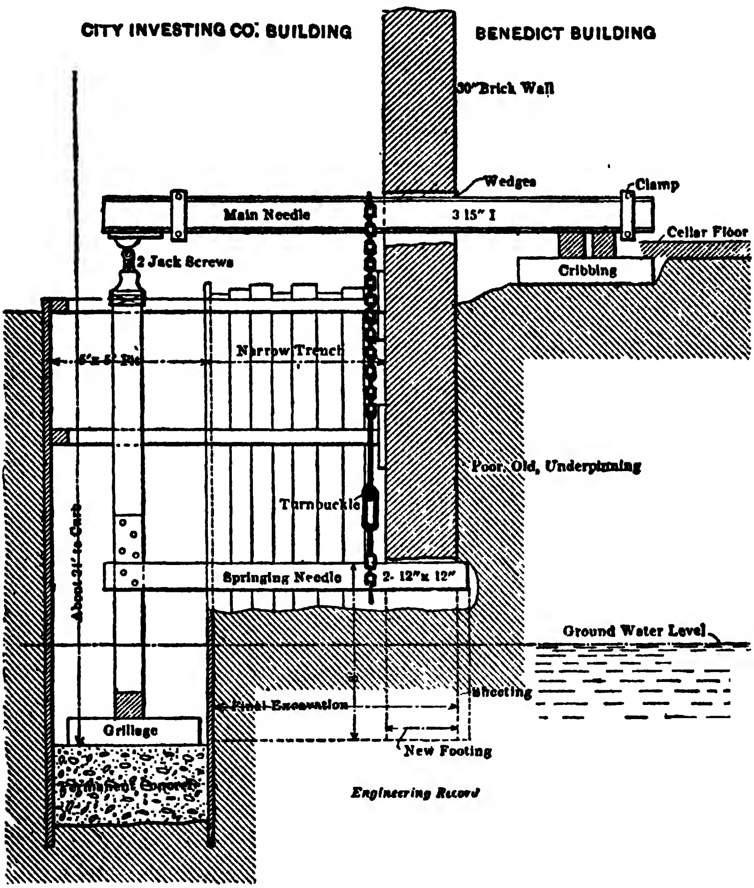


FIG. 166*a*.—Underpinning Methods for Benedict Building, New York.

and the I-beam needles to relieve the connection to the vertical shores and take the upward cantilever reaction."

Figure 166*b* illustrates another method for a suspended support for the footing. [1]"A steel bearing plate was seated across

[1] Engineering Record, vol 56, page 348, Sept. 28, 1907.

the top flanges of each pair of I-beams and gave bearing for the nuts on the upper ends of two 2-inch vertical rods about 7 feet long. The nuts on the lower ends of these rods engaged a cross-plate or saddle, forming a fulcrum for an 8-inch horizontal cantilever I-beam 10 feet long. The long arms of the cantilever reacted upward against some of the I-beam stringers supporting the outer ends of the needle-beams. The short arms took bearings about 2 feet long on the under side of the old concrete footing, supporting it across the thickness of the

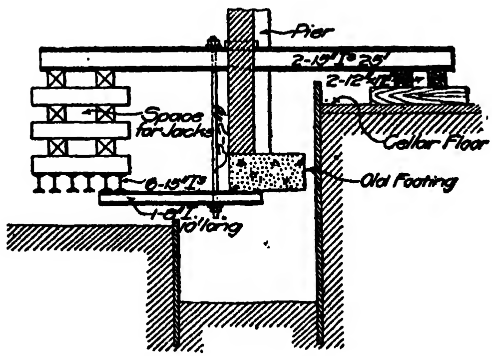


Fig. 166b.—Suspended Support for Footing, Silversmith's Building, New York.

wall, so that when undermined by the excavation for the new foundation the old footing looked in cross-sections like a cantilever projecting about 2 feet beyond the inner face of the wall and proved strong enough to resist the bending moment thus developed."

Art. 167. The Cantilever Method

Where, for some reason, the work cannot be carried on from both sides of the wall, the cantilever method may be employed. The possible modifications of this method are many, but two examples are illustrated to show the fundamental principles. In the construction of the present building at 42 Broadway, New York City, it was necessary to sink caissons close to the

seven-story building then occupying the site of 44 Broadway. In order not to delay the sinking of the caissons, it became necessary to avoid supporting some of the needle-beams on the site of No. 42. For this reason the scheme shown in Fig. 167*a* was adopted.

[1]"Two groups of five 20-inch I-beams, 30 feet long and 21 feet apart in the clear, were put through the foot of the wall

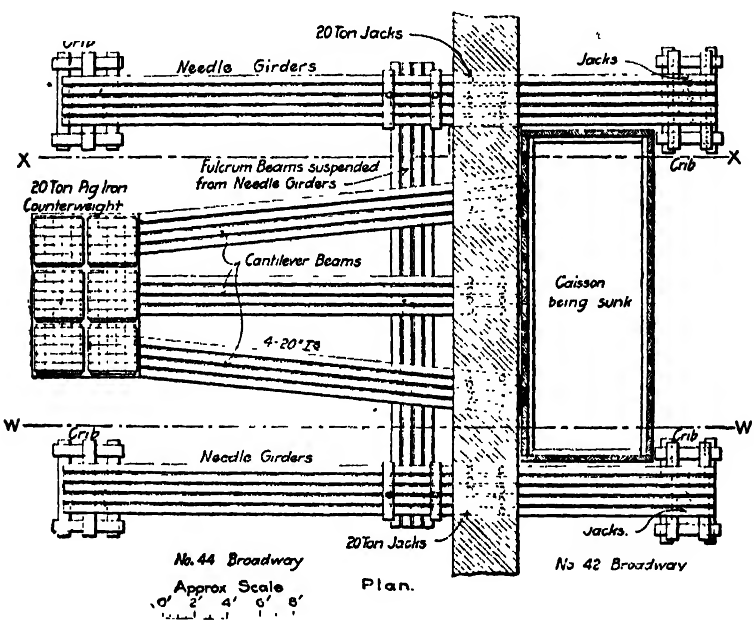


FIG. 167*a*.—Counterweighted Needle-Beams and Girders for Building at 44 Broadway, New York City.

at right angles and supported on cribwork and jackscrews at both ends. In building No. 44 there were suspended from both groups close to the wall four 24-inch I-beams 30 feet long carried on yokes screwed up close to the under side of the needle girders. These beams served as a fulcrum to support three sets of four 20-inch cantilever I-beams each. These cantilevers were located in the center and at both ends of the

[1] Engineering Record, vol. 48, page 698, Dec. 5, 1903.

section of the wall included between the needle girders so as to leave about equal space between them. They converged in No. 44, where a platform was built on their extremities and loaded with pig iron to form a counterweight against the upward reaction. The wall was supported on the needle girders and on the ends of the cantilevers by double rows of special 20-ton jackscrews."

An example of underpinning in which all the supporting was done from the inside, in order not to interfere with construction work carried on outside, is shown in Fig. 167*b*. The needle-beams, with their ends inserted in holes in the wall, were fulcrumed on jackscrews 5 feet from the inside face of the wall. The beams took bearing on blocks of wood which were bored at each end for a 4-inch jackscrew and which rested on cast-steel nuts engaging the screws. The lower ends of the screws took bearing on cast-steel base plates seated on sills which transmitted the load to a timber grillage. Auxiliary supports were wedged up against the needles to take the load in case of failure of the jacks.

The wall loads were transferred to the needles through timber blocks with a few inches of cement on top to develop more uniform bearing. The sets of needles were spaced 9 or 10 feet apart, but, as shown in Section *W*-*W* (Fig. 167*b*) intermediate bearing was obtained through blocking and wedges resting on an 8- by 8-inch horizontal beam. Supporting timbers were jacked up under the lower flanges of the needles in the plane of the wall as a precautionary measure, until the new foundation was ready to be constructed.

The long arms of the cantilevers reacted against the main floor girders in the first floor through blocking and wedges, and were further held down by cast-iron ballast and by anchoring to the piers through pairs of horizontal I-beams. The latter engaged recesses in the piers and had transverse pieces across their bottom flanges. To these transverse pieces were attached lengths of wire rope passing up over the blocks on top of the needles.

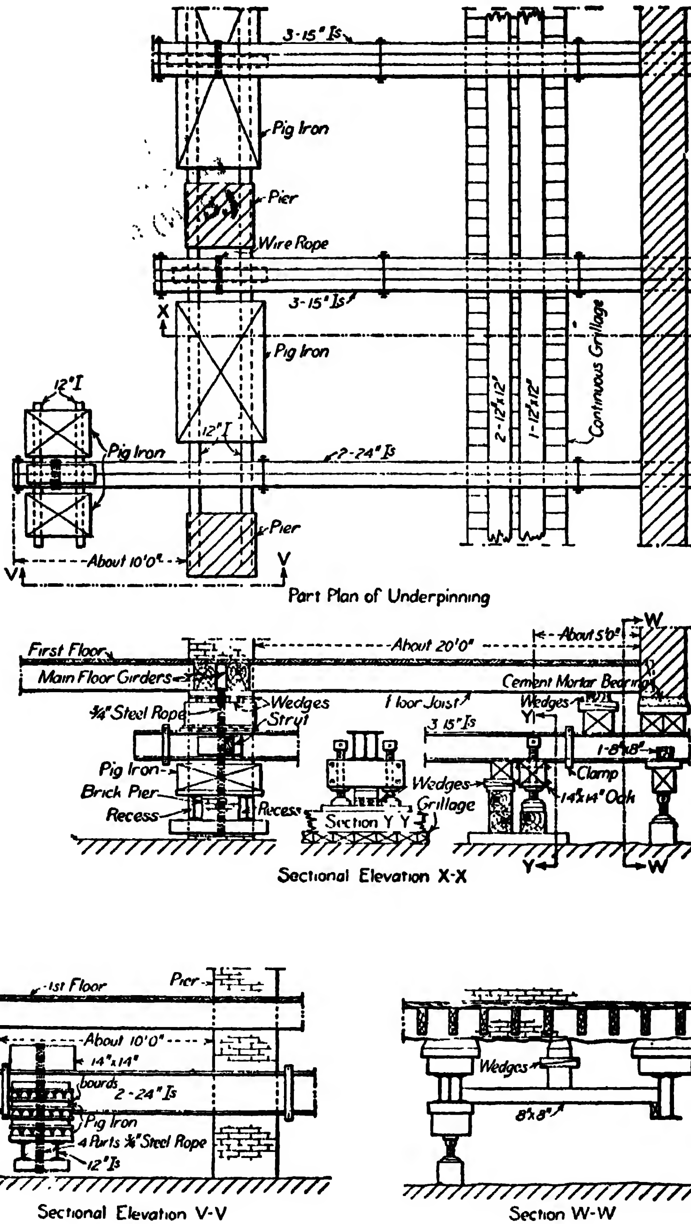


FIG. 167*b*.—Arrangement of Underpinning, 92–94 Maiden Lane, New York City.

ART. 168. FIGURE-4 NEEDLES

The methods explained in the last article are used where it is necessary to avoid using space on the outside of the building. Where there is no available space on the inside, the figure-4 method is generally employed.

Figure 168*a* shows the details of this method as used in the Benedict Building, New York City. Pits 5 feet square and about 6 feet on centers were first excavated and sheeted to about 30 feet below the curb, and on the bottom a 3-foot layer of concrete was placed. On this concrete a timber grillage was erected to distribute the load from a 12- by 12-inch shore to the concrete footing. The lower ends of the shores, which were about 30 feet long, took bearing against short horizontal timbers, the latter in turn bearing against either one or two jackscrews reacting against foot blocks. The upper end of each shore was surmounted with a saddle plate and wedges and was notched into the wall. The saddle plate gave bearing to 1-inch vertical rod suspenders, to the lower ends of which were fastened turnbuckles and chains, engaging the 12- by 12-inch needle-beams.

The springing needles, which by a cantilever action took the load from the wall, were placed by first excavating the space between the sheeted pits and the wall. The outer ends were bolted to the inclined shores and took bearing against reaction cleats above them.

To take the weight of the wall from the old footing, the jackscrews were first operated to bring the shore to a tight bearing to take some of the weight of the wall above, after which the turnbuckles were screwed up until the remainder of the weight of the wall was transferred to the system of needles.

With this type of underpinning, a very stable footing must be provided for the lower end of the shore, for, whereas with the ordinary form of needle underpinning a part of the weight of the wall is transferred to one side and a part to the other, here the entire weight is carried to one side. Another condition to guard against is the tendency of the shore to push in the wall on

account of the horizontal component of its thrust. This horizontal force is ordinarily not large, and almost any building has

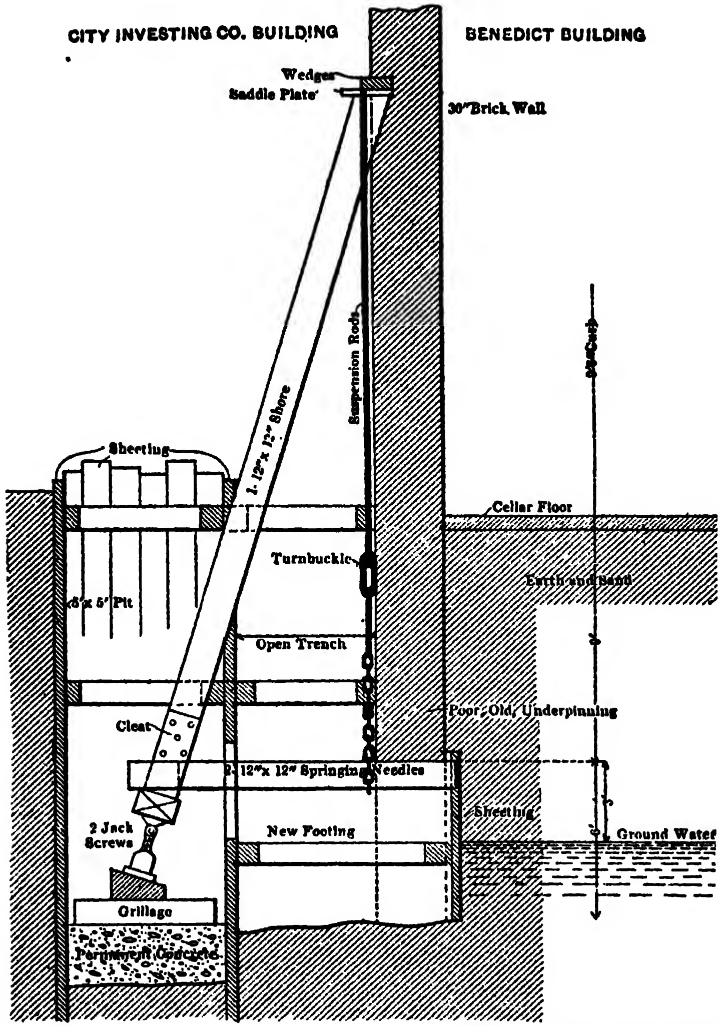


FIG. 168a.—Underpinning with Figure-4 Needle Method, Benedict Building, New York City.

sufficient strength to withstand it if the shore be inserted at a floor level, thus permitting the floor system to transmit it to all parts of the building.

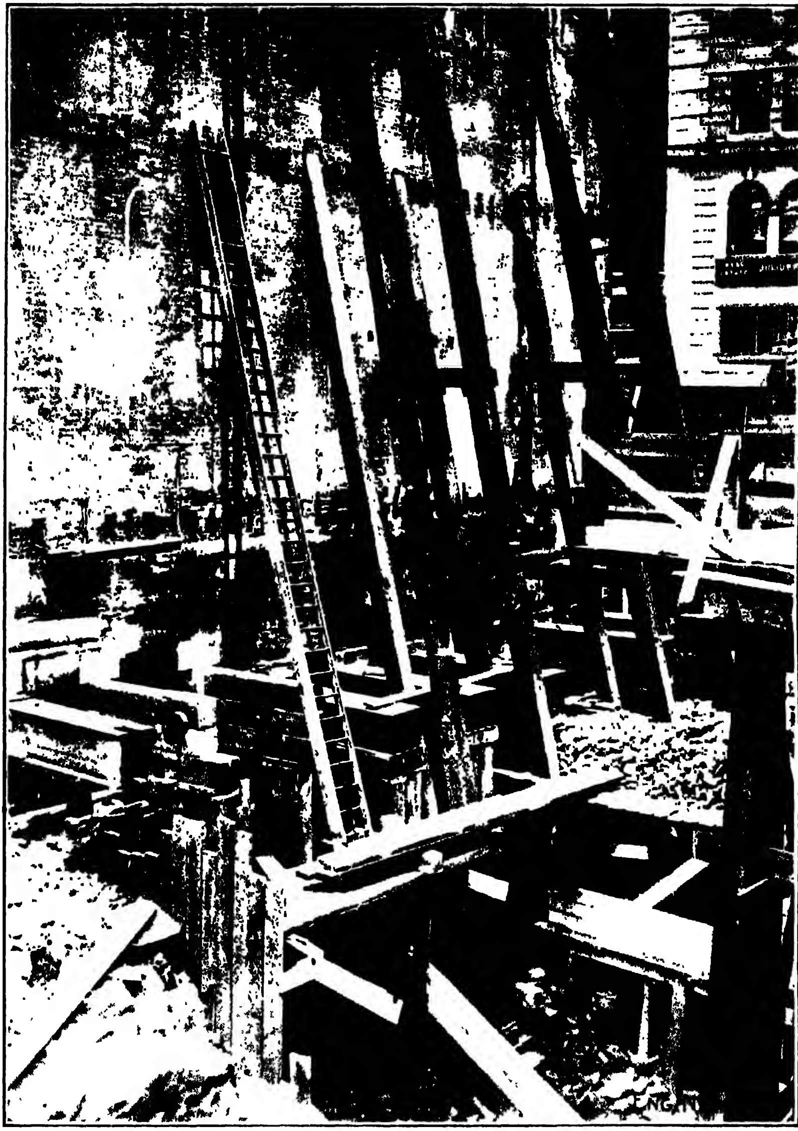


FIG. 168*b*.—Use of Long Shores in Underpinning Cross Building, New York.

(*Facing p.* 552.)

Shores or Pushers.—As an auxiliary to other methods, shores are often used in underpinning work. Fundamentally, they are the figure-4 needle-beams without the vertical rod and horizontal needles, and as a result they can take only the weight of the wall above the points at which the struts are notched into them. For this reason the use of shores must be combined with some other type of underpinning, or else enough

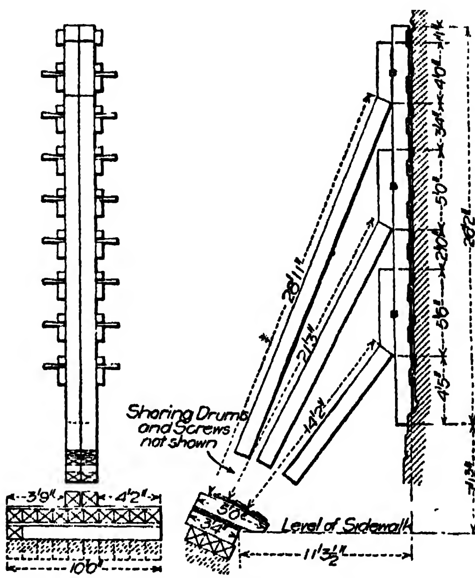


Fig. 168c.—Showing Ashlar Face Wall of the Farmer's Loan & Trust Co.'s Building in New York City.

of the original footing must be temporarily left in place to take the weight of the wall below the shores. Figure 168*b* shows several long shores inserted, while Figure 168*c* illustrates the use of shorter shores.

One of the most successful applications of the use of shores alone was in the underpinning of the Standard Oil Building in New York, where the capacity of the shores was 5 tons. As many as 12 steel screw jacks were used, bearing against a timber mat, the load being carried from the wall through a 24- by 24-inch timber.

ART. 169. PLACING THE NEW FOUNDATION

As the object of underpinning is the protection of foundations from being undermined through the excavation of adjacent material at a lower level, it follows that, in general, only those structures having shallow foundations will require underpinning. The two types of new foundations are the shallow and the deep foundations. The former consists of a simple masonry

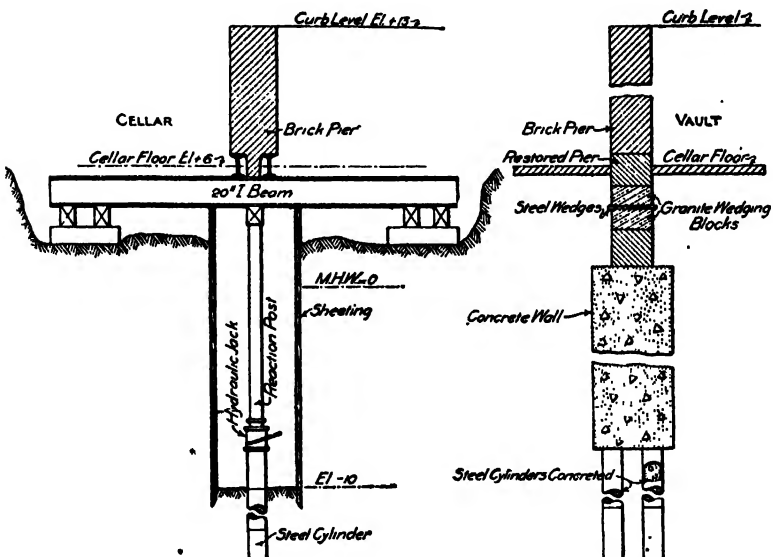


FIG. 169a.—Method of Underpinning Centre Street Buildings, New York, Due to Subway Excavations.

or concrete footing, or of a spread footing, and is the type which uses the needle-beam method of underpinning. The new foundation is placed as deep as the new excavation is to be made. The deep foundation includes the cylinder, the caisson and the tubular pile piers. Masonry and concrete footings, as well as spread footings, are described in Chap. XV.

Where pier systems are used, it is customary to give the wall no temporary support, as the sinking operations deprive the

wall of but a small section of bearing at a time. Conditions sometimes are such that it is advisable to support the wall temporarily on needle-beams before sinking the piers. Such is the case where the wall is weak or is too light to take the cylinder reactions.

Figure 169*a* illustrates a case in which steel cylinders were used. It closely resembles the Breuchaud method described in the following articles, the essential difference being that here needle-beams were used temporarily to take the load.

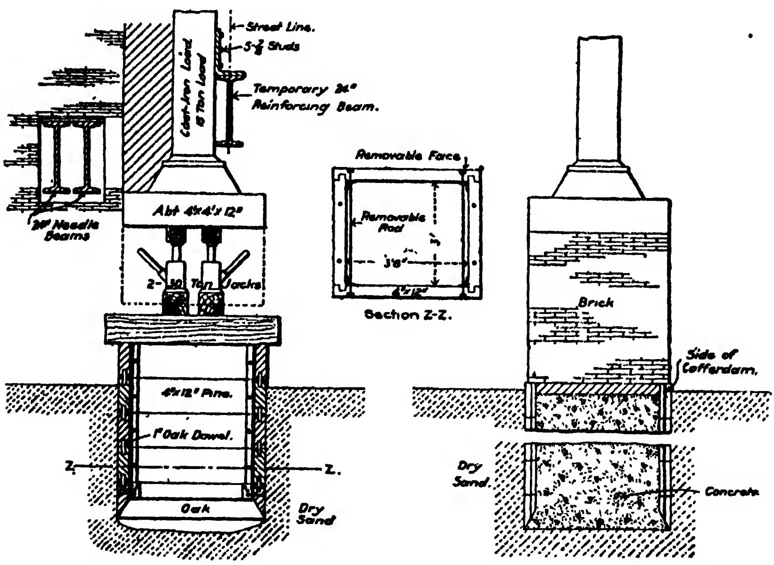


FIG. 169*b*.—Method of Sinking Open Cribs for Underpinning, Merchant's and Trader's Bank Building, New York.

Horizontal I-beams recessed into the face of the wall carried its weight to the transverse 24-inch I-beams, which served the double purpose of carrying the weight of the wall—thus allowing deep trenches to be dug under the same—and of furnishing reactions for sinking the sectional steel pipe by hydraulic jacks. The cylinders were sunk to solid bearing, filled with concrete and capped with a footing wall on which was built the new brick wall to connect to the old wall. The left-hand illustration

shows a single cylinder, and the right-hand illustration two cylinders being used.

Figure 169*b* indicates the use of a caisson for the new foundation of a column. [1] "A heavy bracket made of a steel plate 26 inches long and 1¼ inches thick, bent at right angles, was secured to the street face of the column by five ⅞-inch stud-bolts. Its longitudinal face took bearing on the upper flange of a 24-inch I-beam needle parallel to the street line which was supported on a sill in the sidewalk vault and on cribbing in the excavation for the new building. The granite pedestal was removed, the concrete footing torn out and the excavation carried down a little lower to receive the bottom course of a timber crib or caisson 36 inches wide and 42 inches long inside. This course consisted of four 4- by 12-inch oak planks set edgewise, and having their lower sides beveled to a cutting edge. They were connected at the corners by short vertical angles and had 1-inch tie rods with countersunk heads in the direction transverse to the lot line. The two sides parallel with the lot line had mortised joints with the other two sides, and the frame thus formed was provided with vertical oak dowels 1 inch in diameter, which projected from the upper edge to lock it to the succeeding course.

"Timbers were set on the upper edges of the planks, and two 30-ton jacks seated in the middle of them reacted against the granite pedestal which was inserted between their tops and the base plate. A laborer with a shovel and scoop excavated the sand from the interior of this caisson as it was forced down by two other men operating the hydraulic jacks. When the jacks had made a full stroke, the pedestal was supported by cross-pieces temporarily inserted under it and bearing on needle-beams and sills. The jacks were then released, another course added to the caisson, the jacks replaced, the caisson forced down and excavation continued, and so on." When the caisson was sunk to place, the bottom of the excavation was carefully leveled and the excavation filled with concrete rammed in 6-inch layers.

[1] Engineering Record, vol. 49, page 135, Jan. 30, 1904.

Art. 170. Joining to the Old Wall

After needling the wall and placing the new concrete foundation, brick piers are built upon the new foundation between the needles to within a few feet of the bottom of the old wall. On these piers are placed pairs of cut stones of the same length and thickness as the piers, and about 14 inches high. One stone sets loosely on the other, with pairs of steel wedges between. The brick pier is then continued on the upper stones until the under side of the old wall is reached, to which it is carefully joined. The wedges are then driven together until the load is lifted from the needles, after which the latter are removed and the brickwork of the wall completed, the final appearance being that shown in Fig. 169*a*. The wedges are then sawed off flush with the faces of the wall and the space between the cut stones filled with grout. Theoretically, the entire load is carried through the wedges, but actually some settlement doubtless occurs to distribute the load throughout the length of the wall.

The caisson foundation shown in Fig. 169*b* was joined to the column as follows: [1] "The top of the caisson was covered by heavy flagstones set so as not to bear on the timber walls, and on them a brick pier was built up nearly to the height of the granite pedestal and capped with a cut granite block, between which and the pedestal, pairs of steel wedges were driven until the weight of the column was transferred from the needle beam to the new footing."

Art. 171. The Breuchaud Process

The Breuchaud process of underpinning consists of sinking a series of cylinders in the plane of the wall and spaced a few feet apart. They are sunk to hardpan or rock and form the support for the wall. The work is carried out in the following order: first, horizontal and vertical recesses are cut in the wall near its foot; second, horizontal bearing beams and vertical steel cylinders are placed in these recesses; third, the cylinders are forced down to solid bearing by jacking against the under

[1] Engineering Record, vol. 49, page 135, Jan. 30, 1904.

side of the horizontal beams and by excavating the material form the interior of the cylinder; and, fourth, the cylinders are then filled with concrete and wedged against the horizontal beams, thus transferring the weight of the wall from the original supports to rock or hardpan.

This method possesses the following advantages over the needle-beam method: first, it occupies less space; second, it makes unnecessary entering the basement of the building, the cutting being done from the outside and usually not entirely through the wall; and, third, it is cheaper if the foundation is to be carried down a considerable distance. In general, it may be said that for shallow underpinning the needle-beam method is preferable, while for deep work the Breuchaud method, or a modification of the same, is better.

The Breuchaud process may be divided into two systems, the pipe and the cylinder. The essential difference in the operation of the two systems lies in the fact that workmen can enter the cylinders but not the pipes.

DESCRIPTION OF SHELLS.—The shells are usually made with a cutting-edge section of steel, and with other sections of steel or cast iron. The sections are from 4 to 8 feet in length, and when of cast iron are flange-bolted on the inside; when of steel they may be flange-bolted or fastened with a screw joint.

The load to be carried or the necessity for workmen to enter the cylinder determines their diameter. The magnitude of the load to be taken by a cylinder depends upon the weight of the wall per linear foot and the spacing of the cylinders. The shells are usually designed to take all the load, no reliance being placed on the concrete filling to assist in carrying it, since the load is placed on the cylinders before the concrete hardens. The spacing of the cylinders cannot vary between wide limits, for, on the one hand, there must be a clearance between cylinders sufficient to furnish proper bearing area on the soil while the piers are being sunk, while, on the other hand, the maximum spacing is limited, owing to the local concentrated stresses involved in carrying the load from the wall to the

cylinder. The usual spacing of cylinders is from 5 to 12 feet. The cylinders may vary in diameter from about 6 inches, carrying a load of from 30 to 40 tons each, to 3 or 4 feet, carrying a load as high as 400 tons. A double-shell cylinder is sometimes used, in which case added strength is developed by breaking horizontal joints.

If workmen are to enter the cylinder, it should have a diameter of at least 30 inches. This will be desirable where boulders are encountered, where sinking must be done through hardpan or where it is desired to prepare the bottom carefully on completion of sinking. The work in the cylinder must usually be done under air pressure and hence they are made so as to be easily transformed into pneumatic caissons.

Figure 171*a* shows the lower riveted steel section of the cylinders and the air-lock used in underpinning the Stokes Building, an 11-story structure having a wall load of 45 tons per linear foot. The top of this section was faced to receive the bottom of the lowest of the cast-iron sections, which were made in 6-foot lengths ordinarily, with special 2- and 4-foot lengths. The flange connections were made with 28 1-inch steel bolts, and were machine-faced to give a clearance in order to prevent bearing of flanges, so that the pressure would be transmitted directly through the shells of the cylinder. The cylinders were built up to the required height and on the upper sections were placed the steel-plate bearing rings to receive the girders on which rested the jacks. The 5-inch pipe carried away the material washed out.

On reaching hardpan, the bearing ring was removed and a heavy cast diaphragm was inserted between the flanges of the last two sections. A similar one was also placed on top of the upper section. The top section was then made to serve as an air-lock by fitting steel doors, with rubber gaskets, to the under sides of the diaphragms.

The cylinders used in underpinning the Trust Company of America Building, New York City, were made entirely of steel and were 3 feet in diameter, with two longitudinal lock-bar joints. The metal was ⅜ inch thick. Each section had

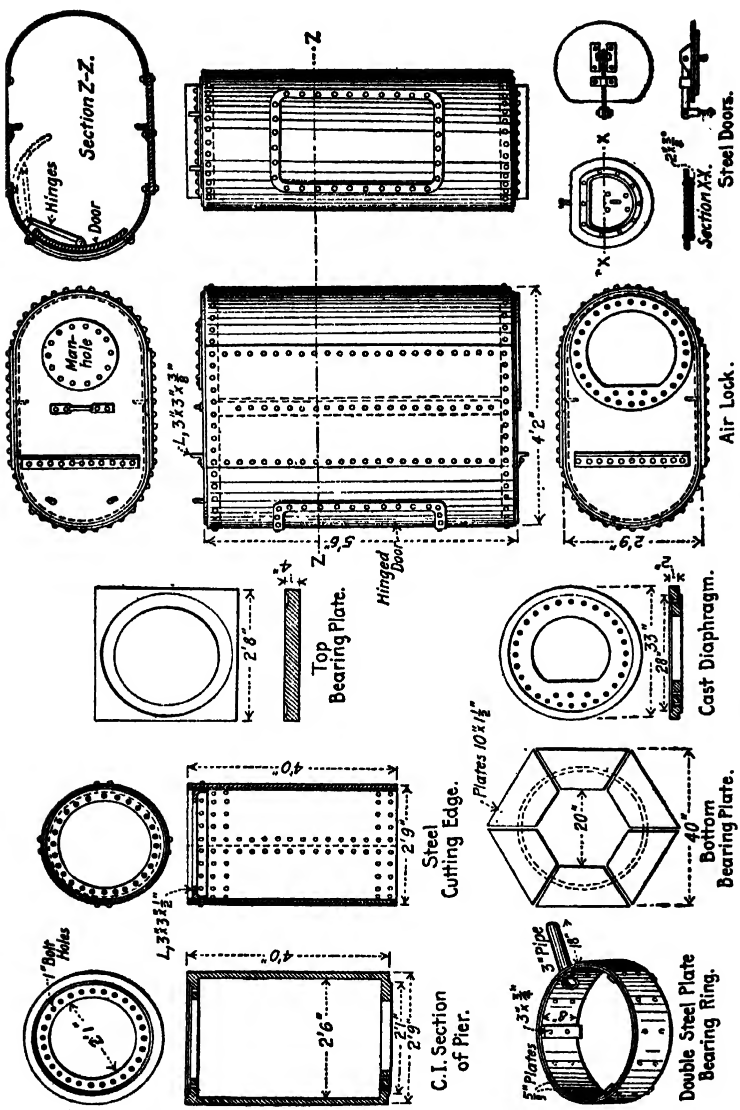


FIG. 171a.—Details of Lower Section of Cylinder, Etc., Used in Underpinning the Stokes Building, New York City.

horizontal circular angles riveted to it at both ends to provide flanges for connecting the successive sections.

In some underpinning at 73–75 East 54th Street, New York, cylinders of ⅜-inch steel and 12 inches in diameter were used. They were first set up in 20-foot lengths. [1] "As these were jacked down, the upper sections, 4 feet long, were coupled to them by inside cast-iron sleeves about ⅜ inch thick and 9 inches long, slightly tapered at the ends to enter the pipe and having a horizontal exterior rib, about 1 inch wide, and of the same thickness as the pipe, on the center line. The edges of the rib were slightly beveled so that the ends of the pipe would draw up against it and make a solid contact under heavy pressure, thus insuring a perfect fit and very rapid assembling of the pipe sections as the work progressed."

ART. 172. METHOD OF SINKING CYLINDERS

The cylinders are sunk: first, by means of hydraulic or screw jacks bearing against the wall above and forcing them down; second, by using a water-jet on the inside of the cylinder to loosen the material around the cutting edge; and, third, by excavating the material from the inside.

A horizontal recess of a size sufficient to receive I-beams is first made in the wall about 12 feet above its base. I-beams are then placed in it and wedged up tightly against the top, after which a vertical recess, extending from the horizontal recess to the foot of the wall, is cut out. The horizontal beams serve the double function of carrying the weight of the wall above the vertical recess and acting as a reaction for the jacks used in sinking the cylinders. On completion of the vertical recess, the lower section of the cylinder is placed in position in the recess. A screw or hydraulic jack is then placed on the section and forces the cylinder down by reacting against the horizontal I-beams through blocking. As soon as the cylinder has been forced down a distance equal to the full stroke of the jack, more blocking is placed between the latter and the bearing beams, and the operation repeated. On completion of the sinking of one section of

[1] Engineering Record, vol. 64, page 276, Sept. 2, 1911.

the cylinder another section is added and the operation repeated until the cutting edge has reached the desired position.

The larger cylinders fitted for pneumatic pressure are sunk as far as possible by washing out the material, assisted perhaps by a sand pump; the doors are then put on and the remainder of the sinking is done by the pneumatic process. At the same time jacks are operated to force the cylinder down. In the smaller pipes the material is sometimes bored out with an auger, a 9-inch auger being used for a 10-inch pipe.

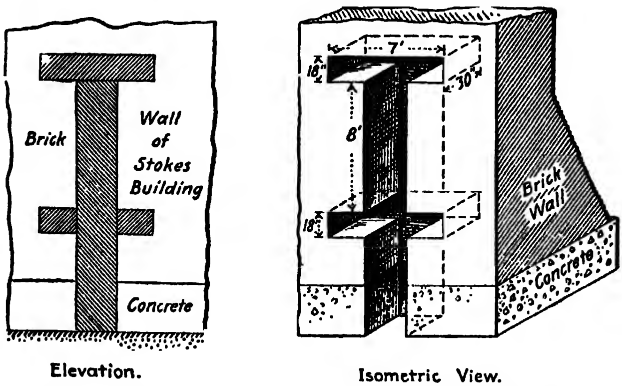


FIG. 172a.—Recesses Cut in the Wall for Underpinning.

The following description, together with that given in Art. 171, shows the method used in placing the cylinders for the Stokes Building, this being the first large structure in which the Breuchaud method of underpinning was applied, the work being done in 1896. Recesses of the form shown in Fig. 172*a* were cut in the wall to a depth of 3 feet, the material in the upper horizontal rectangle being first removed. Five 15-inch I-beams were then placed in this opening and wedged tightly against the upper surface of the recess. The vertical rectangular recess was then cut out. A section of the cylinder was then placed in the vertical recess and on it were placed two I-beams 34 inches long. On these I-beams rested a large hydraulic jack, which took bearing on the 15-inch I-beams above through timber blocking. As the cylinder was jacked down the material inside was washed out with a jet pipe.

ART. 173. CONCRETING THE CYLINDERS

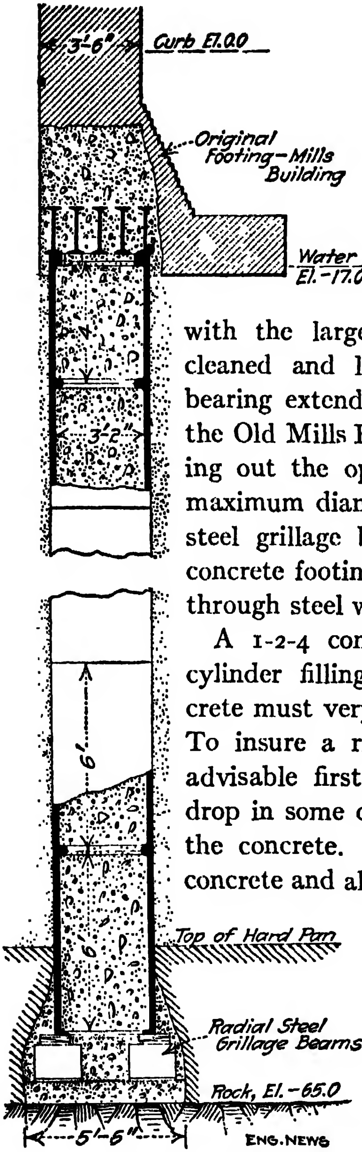


FIG. 173a.—Typical Underpinning Cylinder for Mills Building, New York.

Where pipes are used little can be done in preparing the bottom further than to inspect it; to pump out the water if possible; and to see by means of an electric light if the desired bearing has been reached and that all loose material has been removed. On the other hand, with the larger cylinders the bottom can be cleaned and leveled off, and the foundation bearing extended if desired. This was done in the Old Mills Building of New York, by spreading out the opening through the hardpan to a maximum diameter of 5 feet 6 inches. Radial steel grillage beams (Fig. 173*a*), resting on a concrete footing, took the load from the cylinder through steel wedges.

A 1-2-4 concrete is ordinarily used for the cylinder filling. For pipe cylinders the concrete must very often be placed through water. To insure a rich mixture at the bottom, it is advisable first to pump in some grout or to drop in some dry cement, and on this to place the concrete. After placing a few feet of the concrete and allowing it to harden, the pipe may be pumped out and the remainder of the concrete laid in the dry. Where deposited through water, a cylindrical bucket of a diameter somewhat smaller than that of the pipe and about 3 feet long is often used. The bucket has a flap bottom, and two lines, one

attached to the bail of the bucket and the other to the flap. In lowering the bucket, the weight is carried by the flap line, but after the bucket is seated on the bottom it is pulled up by the bail line, which causes the concrete to be deposited through the bottom. This avoids any possibility of the water washing the concrete and separating the constituent materials.

For pneumatic cylinders the working chamber is first filled with concrete, with perhaps a layer of grout or mortar on the bottom, after which the air pressure is left on for about 48 hours. The remainder of the cylinder is then filled with concrete.

ART. 174. TRANSFERRING LOAD TO CYLINDER

Figure 174*a* illustrates the method used in transferring the loads to the cylinders in the Empire Building, New York. The underpinning cylinders were first capped and the recess above them filled with brickwork to a certain height. On this brickwork two granite blocks were placed, one resting loosely on the other, after which the remainder of the brickwork was placed. Pairs of steel wedges were then inserted between the granite bearing blocks and driven together, thus separating the two blocks and bringing the wall loads to the cylinders. The space between the blocks was then filled with cement grout.

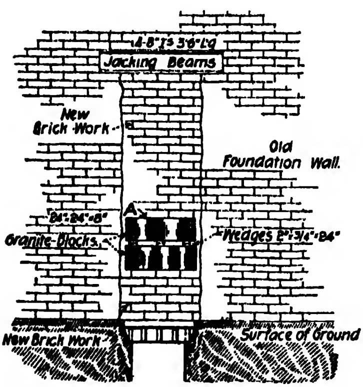


FIG. 174*a*.—Underpinning the Empire Building, New York.

After the cylinders of the Stokes Building had been filled with concrete, the top of each cylinder was capped with a top bearing plate (Fig. 171*a*). Five I-beams were then placed on the top of each cylinder, as shown in Fig. 174*b*. Steel posts were placed in the vertical recess and rested on steel plates, which, in turn, rested on the lower tier of I-beams, the posts extending to within 2 inches of the under side of the upper tier

of I-beams previously placed (Art. 172). Steel plates were placed on these posts and pairs of forged-steel wedges then driven together between these plates and the lower surface of the upper tier of I-beams to bring the wall load to the cylinder. After this the recess was solidly bricked up.

The lower tier of I-beams serves a triple purpose: first, it reduces the amount of load coming through the upper tier, thus lessening the stress in the brickwork at that point; second, it carries the weight of the wall below the upper tier of beams to the pier, where it must otherwise remain on the old footing; and, third, it eliminates stress in the new brickwork in the vertical recess.

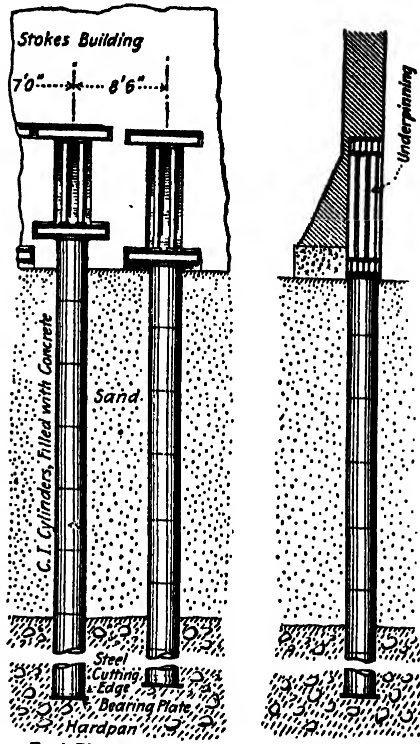


FIG. 174b.—Underpinning the Stokes Building, New York City.

ART. 175. PRETEST PILE UNDERPINNING

In the work of underpinning about 75 buildings on Williams Street, New York, in 1915, the Breuchaud method was considerably modified in a number of respects. The diagram of Fig. 175a shows the typical method used under a three-pier foundation in the front of a building. Two I-beam or latticed girders were clamped around the outer and inner sides of the foundation slabs and bound together with cables between the piers. The transverse faces of the slabs were also fitted into channels or I-beams, whose ends rested on the longitudinal

girders, after which the whole was concreted to form a solid mass.

A pit to ground water between the columns was then dug and sheeted. A 2-foot secton of a 14-inch pile of ⅛-inch steel was then placed in the pit bottom and jacked down by an hydraulic jack. When completely driven, another section was added and the operation repeated. As the pile was sunk the material in the same was removed by a minature orange-peel bucket, a helical screw or other simple method. When all the piles of

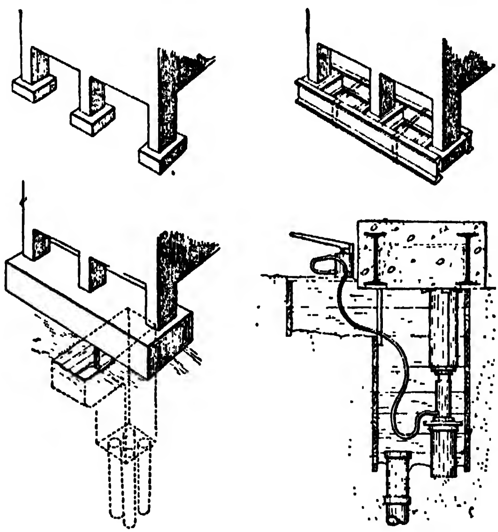


FIG. 175a.—Pretest Pile Underpinning.

one foundation were placed, each shell was filled with concrete and the pile tested with an hydraulic jack to a load at least 50 percent greater than that apportioned to it, about 33 tons; being jacked down, if necessary, until it would carry that load without settlement While all piles were carrying this 50 percent overload, a column of steel shapes was erected on each pile and wedged against the concrete above by steel wedges. The whole pit was then concreted and grout introduced under a 4-foot head to fill all voids and take up any shrinkage.

The pile lengths were extremely strong for a hollow shell only ⅛ inch thick. In one case a shell was jacked down to gravel and with its top flush with the ground sustained a load of 89 tons. The joints were of the butt type with a 2-inch inner lap and an outside reinforcing band.

By the use of this method there is a definite assurance that every pile is amply strong to take its share of the load, and by transferring the building load to the piles while they are under heavy stress only a very slight settlement of the building results.

This method has been used in the construction of new buildings as well as in the underpinning of old structures, the piling being carried down as the building is erected, this effecting a considerable saving of time.

ART. 176. OTHER MODERN METHODS

Another method of underpinning, developed to cheapen the cost and reduce the risk in many cases, especially where the building to be underpinned is light or poorly constructed is to sink shafts with plates, as was done in underpinning the Cambridge Building, New York City.

[1] "The cylinders (Fig. 176*a*) were 4 feet in diameter, and had very thin shells. The method of placing them constitutes a new system of underpinning; for, instead of placing the bottom section first and jacking it into the ground, and then placing another section on top and repeating the operation, a space was excavated for the top section, which was made 4 feet deep, and the space outside of this top section was back-filled, generally with concrete, and remained permanently in that position.

"A laborer then entered the top section and, with a short-handled shovel, excavated the material under the cutting edge sufficiently to insert one segment of the next ring. All the rings below the top section were 2 feet deep and in four segments, one of which was small, to act as a key, as in tunnel lin-

[1] Underpinning the Cambridge Building, New York City, by T. K. THOMSON, Transactions of the American Society of Civil Engineers, vol. 67, page 553.

ing. In this key segment the connection angles were bent 13½ degrees from the radial, to permit putting the key in place. The joints of the other segments, of course, were on the radial

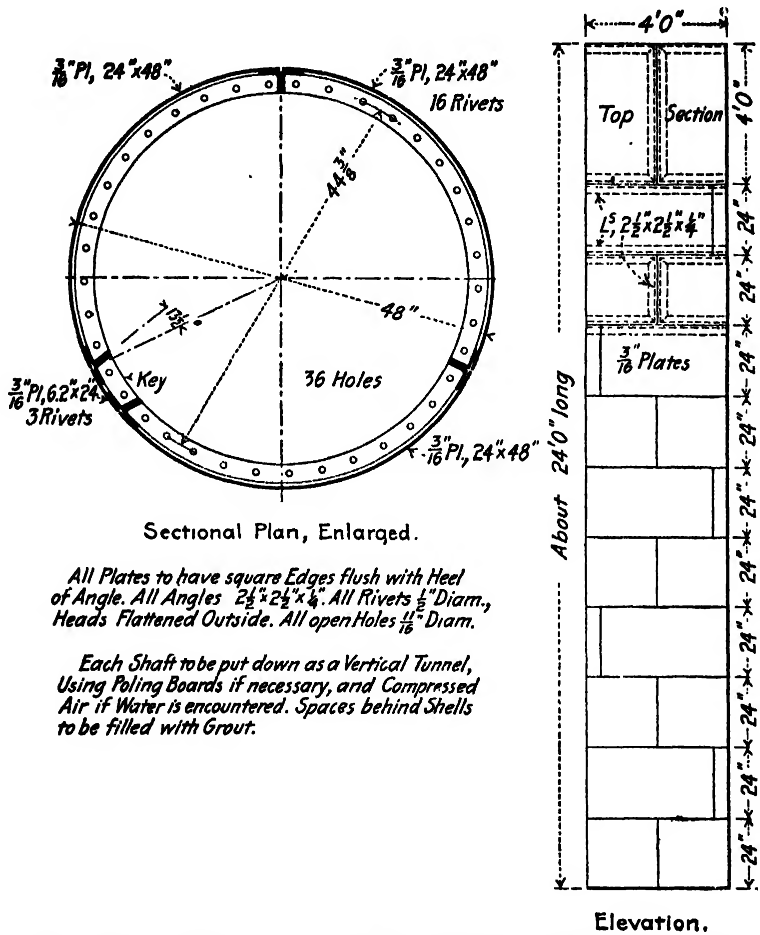


FIG. 176a.—Underpinning by THOMSON'S Method of Vertical Tunneling.

lines. All the plates were 3/16 inch thick, and all the angles, both horizontal and vertical, were 2½ by 2½ by ¼ inches. When the first segment under the top section had been bolted in place, the excavation was made for the second segment, then

the third and then the fourth or key. Where the ground was good, the excavation was made for all four segments at once." No water was encountered in the above operation, thus eliminating the necessity for using air pressure. Where the latter is used, all joints must be calked.

Another method, patented by OGDEN MERRILL, makes use of a nest of telescoped cylinders or caissons. The upper section

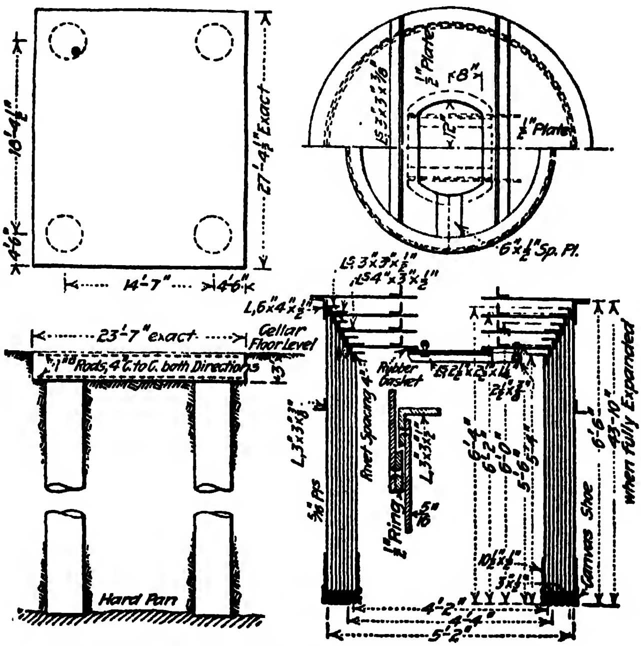


FIG. 176b.—Merrill Telescopic Method of Underpinning.

is first jacked down and then the other sections are jacked inside of this, after removing the material from the inside. The pneumatic process has been applied also to this form of construction.

This method is more expensive than the THOMSON one, but it is safer in that it avoids the necessity of excavating below the cutting edge to insert lining segments.

An example of this type of caisson is illustrated in Fig. 176a, there being eight sections, the bottom one 4 feet in

diameter and $4\frac{1}{3}$ feet high. Each consecutive section is 2 inches less in diameter than the one above it and, as the shell plates are $\frac{5}{16}$ inch and the edging bands $\frac{1}{2}$ inch thick, the clearance on each side is $\frac{3}{16}$ inch. On the inside of the bottom of each section is riveted a band of 3- by $\frac{1}{2}$-inch steel plate with the top edge beveled backward, adapted to engage the beveled edge of a steel angle riveted around the top of the next inner section, this preventing the sections from pulling apart.

CHAPTER XVII

EXPLORATIONS AND UNIT LOADS

ART. 177. TEST PITS AND SOUNDING RODS

A simple method of examining a foundation site is to sink test pits by open excavation somewhat deeper than the excavation otherwise required for the substructure. Sometimes it may be necessary to line the pit with sheeting and to use a hand pump to keep down the water, but, instead of increasing the cost materially on this account, other methods may be adopted. The principal advantage of the open pit consists in the possibility of examining not only the variation in character of the earth encountered but in observing the degree of its natural compactness when in place. This method is practically confined to shallow foundations.

A sounding rod consists of an iron rod or pipe, 1 or 1½ inches in diameter, which is driven into the ground with a maul, and turned after each blow. It serves merely to determine whether the resistance is increasing or decreasing, variable or constant. A sunken log, boulder or some other obstruction can stop the driving. The information thus secured is so unsatisfactory and inadequate that it is frequently misleading. The rod may find a stratum of gravel but fail to reach the soft stratum underneath it, because the resistance is so large that it cannot be driven farther. In one instance, eight men on the handle bar were unable to push the sounding rod over 7 feet into stiff mud or clay, but on driving test piles at the same spot no difficulty was found in driving down 70-foot piles.

Figures 177*a* and *b* show the details of a more elaborate sounding-rod outfit, used for finding the location of rock to a maximum depth of 30 feet along the line of the Welland Ship Canal.

Where the drill rods could be withdrawn without much trouble, solid points were used, but for deep drilling the rods had removable points in the form of cast-iron sleeves, these sleeves remaining in the ground on withdrawal of the rod. In the illustration 2-inch rods are shown, but in the work 1½-inch rods proved just as satisfactory. The hammer weighed 125 pounds.

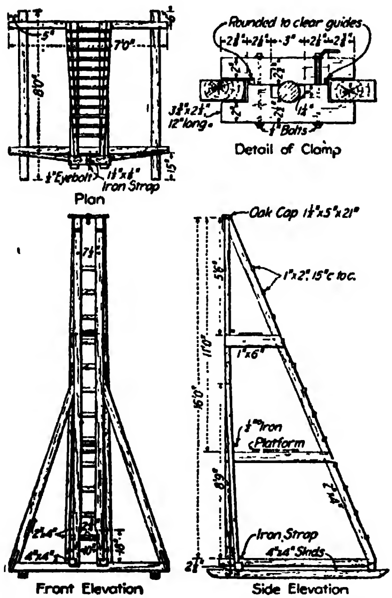


FIG. 177a.—Light Leads of Drill Outfit Built to Drive and Pull Sounding Rods.

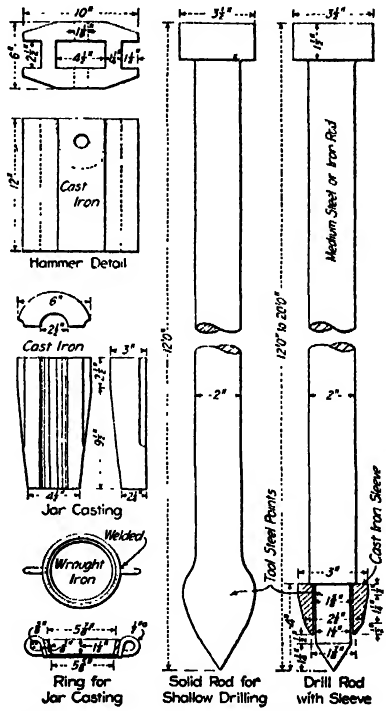


FIG. 177b.—Details of Drill Rods and Hammer for Light Sounding Outfit.

ART. 178. BORING WITH AUGERS

A simple and effective tool to use in exploring the site under proposed structures consists of an ordinary wood-auger 2 inches in diameter welded to a black pipe about 18 inches long, which is connected by ordinary couplings to 12-foot sections of pipe.

The handle used in turning is 2 feet long and is made of two pieces of ¾-inch round iron welded to a strong ring that will pass freely over the coupling, and is secured to the pipe by a ⅝- by 2½-inch set-screw.

In starting the boring great care is necessary to keep the auger vertical. Five turns fill the bit, which is then withdrawn to the surface, and cleaned after examining the ma erial. When the hole is too deep to raise the auger by hand, it is lifted by a block and fall suspended from a wooden folding tripod, a 3-foot chain being used to grip the pipe. Before lowering the chain for a new grip, the handle is attached, so that there is no chance of the auger dropping when any sections of the pipe have been removed.

Another auger 1 inch in diameter connected to 6-foot sections of ½-inch pipe is required when the hole becomes clogged so that the larger auger can no longer be used. With dry sand it is necessary to pour enough water into the hole to make the grains stick together so that they may be lifted. When sand and gravel become troublesome and the hole will not retain its shape, a 3-inch casing is driven, being handled in 4- or 5-foot lengths. A 2-inch drill with chisel point attached to sections of pipe, like the auger, may also be needed to cut through some obstructions. Borings with augers have been used for depths up to 100 feet.

The borings are regularly inspected as the ground is penetrated and a record kept of the depths and variations of materials encountered. Even with the aid of the casing this method of boring is not applicable in fine-running gravel or in quicksand, unless it is a thin stratum. The loose material may then be removed by a sand pump, which consists of a narrow cylindrical bucket with a cutting edge at the bottom and above this a flap valve opening upward. It is partly filled by rapidly raising and dropping it alternately.

One form of earth or clay auger has two cylindrical cutters pointed and bent at the bottom so as to draw the auger into the earth, and, after rotation, to support a lot of excavated material during its withdrawal to the surface. This tool is

designed to penetrate compact material like hardpan and frozen earth.

Sometimes the hole will retain its shape better if the diameter is larger. It is claimed that a diameter of 8 inches is frequently advantageous for ordinary depths unless the proportion of sand is too large. A post-hole digger may be used up to about 16 feet.

For soft material at considerable depths the device shown in Fig. 178*a* has proved satisfactory. It consists of a brass core

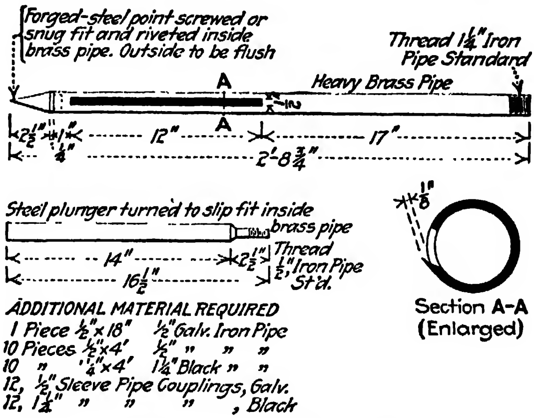


FIG. 178*a*.—Shaw's Special Soil Sampler.

barrel at the end of a 1¼-inch pipe inside the core barrel, there being a steel plunger at the end of a ½-inch pipe. The instrument is forced down into the ground and successive lengths of 1¼- and ½-inch pipe added until the desired depth is reached. The plunger is then drawn up about 2 feet and the larger pipe rotated, the lip of the brass pipe cutting off a sample.

For some work done in 1913 with augers of 3- by 3/16-inch twisted steel, one turn in 6 inches, the cost per foot was approximately 13½ cents. The depths were from 15 to 20 feet, the total amount of work being 6200 feet.

ART. 179. WASH BORINGS

When considerable work is to be done a standard outfit for wash-drill borings is employed which consists of a small der-

rick or tripod, the casing, hollow drill rods, and a hand force pump, together with their accessories and necessary tools. A convenient size of tripod has timber legs 3 by 4 inches in section and 18 feet long. The rope for raising and lowering the casing and drill rods is manipulated either directly by hand or with the aid of a drum. The casing is composed of extra-heavy pipe in about 5-foot lengths, with flush joints so as to form a smooth exterior surface. The usual size has a nominal inside diameter of 2½ inches. The hollow drill rods are made of heavy seamless steel tubing usually in 5-foot lengths, but sometimes as long as 16 feet, connected by special coupling pieces about 6 inches long. The usual size of drill rod has an outside diameter of 1⅞ inches for a 2½-inch casing. To the bottom drill rod is attached a chopping bit with an X-shaped chisel point, and with four openings for the water-jet. The hand pump consists of a double-acting force pump with a single hand lever and 1½-inch suction. The upper drill rod is connected to the hose from the pump by means of a hoisting water swivel, so that it may be raised or lowered and rotated during operation without twisting the hose.

During operation, water is forced down through the hollow drill rod, and, escaping through the jet at the bit, carries upward the loose material in the annular space between the rods and casing. In suitable material the drill rod is worked down by rotating it. In harder material it must be lowered by "churning." This is accomplished by raising it up a short distance and letting it drop. Meanwhile, the casing is also worked down by rotating it. If this will not answer the purpose, the casing must be driven down. If the tripod has only a single pulley the drill rods have to be withdrawn when the casing is to be driven deeper. By using a double block and a jar weight, the operation of driving the casing may go on simultaneously with the drilling and jetting. In this manner the casing is kept loose. To protect the casing, a drive head is screwed into the top section and into that a hollow guide which guides the cylindrical jar weight in its movements. The upper section of the casing has slots in the side for the escape of the water and

sediment or other loosened material. After the overflow which is caught in a bucket has settled, samples are taken and put in glass bottles for preservation. They are properly labeled to correspond with other records.

This operation is continued until the required depth is reached or until obstacles like boulders are encountered. If a boulder is not too large, a small charge of dynamite may be used by an experienced operator, care being taken to raise the casing high enough to avoid any damage from the explosion. With this equipment, borings can be made in sand, gravel, clay in varying degrees of hardness, including indurated clay, and hardpan. For borings into rock, shot drills or diamond drills are required.

When only a small amount of work is to be done and in light or sandy material, a less expensive outfit may be employed. Ordinary pipe can be used for the casing, a good size being 2 inches in diameter. Gas or water pipe $\frac{3}{4}$ inch in diameter may be substituted for the hollow drill rods, with a water swivel made of ordinary bends and nipples. Lighter chopping bits may be used. Sometimes the bit has only a single chisel point with an opening on each side for the jet, or the bit may be replaced by a plain jet pipe. In silt and some mixtures of clay and sand, it may not be necessary always to have the casing follow the inner pipe to its full depth, as the material will retain the shape of the hole. In sand and gravel both pipes must be sunk together, pains being taken to keep them turning and occasionally to lift the inner pipe a little to prevent it from binding. In some instances the casing has teeth cut into the end of its lower section, which is also flared out slightly. This will facilitate sinking by rotation, but little, if any, hammering being required.

Since only the finer material may be washed up while the coarser material is pushed aside in the hole which is scoured out by the jet, it is possible to misinterpret the indications of wash borings. In order to obtain samples of the material penetrated in its natural relation, the bit or jet may be replaced temporarily by a short piece of brass pipe, which is then

pressed into the softened ground at the bottom and lifted out for examination.

It is not always possible to distinguish between a large boulder and bed rock. By making a number of borings on different parts of a foundation site and comparing the elevations of the supposed rock surface, it may be assumed that, unless these are nearly at the same level, the higher erratic elevations may indicate boulders. Additional borings should be made near these locations to see whether greater penetrations can be obtained.

Since the action of the jet and of the chopping bit often radically changes the natural condition of some material penetrated, it is desirable to take out dry samples or cores whenever feasible. This can be done in the hardest clay or the softer shales by using a saw-tooth bit working dry and thus obtain a perfect knowledge of the material. Unless this is done a hard clay which is suitable in every way for a foundation bed may be passed by, and thus incur unnecessary extra expense to carry the foundation to a lower level.

Although in some cases the results obtained by wash borings alone may be only negative, their use in conjunction with core borings for the balance of the depth required may save expense by materially reducing the number of core borings.

The records of test borings should show the kinds of material, the thickness of various strata, the elevation of ground water, etc. By comparing these data for various parts of the site, it can be seen whether any given stratum is fairly uniform in thickness or runs out between two borings, and which stratum should be selected to bear the given load, or to receive some further test by loading if necessary.

As an illustration of the information furnished by borings and what its effect was upon the construction of the foundation, reference is made to Engineering News, vol. 68, page 914, Nov. 14, 1912. The site of the Brooklyn anchorage pier of the Manhattan bridge was explored by nine preliminary test borings which indicated sand, gravel, boulders and clay in irregular strata down to rock at a depth of 70 to 75 feet below mean

high water. Tests of the ground were made by loading it and by driving piles, and from a study of all these results and the surrounding conditions it was decided to drive bearing piles to carry the foundation load.

An analysis of cost for wash-drill borings made in residency No. 5 of the New York State Barge Canal may be found in a valuable article by EMILE LOW in Engineering News, vol. 57, page 54, Jan. 17, 1907. The analysis includes 14 items of cost, and the total cost per linear foot of boring for 17 monthly accounts and several field parties ranges from 17.55 to 52.63 cents, the average being 36.97 cents. The average depth of hole is 27.7 feet. The cost given does not include that of the plant nor that of extraordinary repairs, for which an addition should be made estimated at 2 cents. The cost of the outfit and tools required for each party was $277.98. The article gives a full description of the equipment, copies of the forms used for records and reports, and the character of the material penetrated for each account.

The costs for similar borings made on the Deep Waterways Surveys in the same state during 1897 to 1900 are 11.2, 13.0, 25.1, 54.1, 68.4, 70.6 and 85.2 cents per foot for different routes or divisions. The borings varied in depth from a few to 190 feet.

ART. 180. CORE DRILLING WITH DIAMONDS

Rock strata are tested by using core drills to remove specimens of the rock in the form of cores which can be examined. In some of these drills the cutting is done by black diamonds, in others by chilled shot or crushed steel and in still others by toothed cutters.

In the operation of a diamond drill a hollow bit is rotated rapidly, in which two rows of diamonds are set around the edge in such a manner that all the cutting is done by them and with a small clearance both inside and outside. Water is forced down through the hollow drill rods and bit to keep the latter cool, and in passing up outside of them carries the cuttings to

the surface. The bit is screwed to the core barrel and that in turn to the drill rods. The bit cuts an annular channel and the core formed within it is protected by the descending core barrel. At intervals, the core is broken off by a special device and lifted out along with the barrel. Figures 180a[1] and b show the core barrel, bit and lifter of a standard make of drill. Two kinds of black diamonds are used for this work, carbons being set for cutting hard rock, and bortz for soft rock. The bortz is as hard as the carbon but not so tough. For medium rock, half carbon and half bortz may be used.

FIG. 180a.—Core Shell, Bit and Lifter of Diamond Drill.

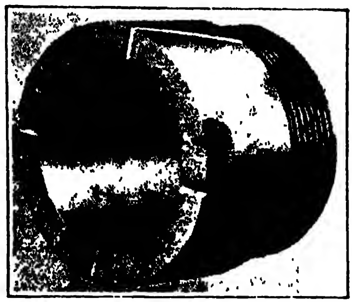


FIG. 180b.—Diamond Bit.

In practice it is customary to let diamond-drill work for foundations by contract, as operators of skill and experience are required. Drilling machines are manufactured of different designs and in a number of sizes, operated in most cases by power. The diameter of core varies from 1 to 2 inches for foundation purposes, larger sizes of core being removed in wells, tunnels and deep-mine prospecting.

If surface material overlies the rock, it must first be penetrated by the method of wash borings as described in the preceding article and the casing driven into the surface of the rock. making a tight joint to exclude the entrance of sand or clay. The diamond drill may then be set up over the casing and operations continued.

[1] Courtesy Sullivan Machinery Co.

The following statement is quoted from an article by resident engineer F. H. BAINBRIDGE in Mine and Quarry for October, 1908, regarding the borings made for the Chicago and Northwestern Railway bridge over the Mississippi River at Clinton, Iowa. The equipment was mounted on a scow. The diamond drill operated 2-inch core bits. Both standpipe and casing had flush joints, their diameters being 4½ and 3 inches, respectively.

"The materials encountered were in order as follows: recent alluvial sands, glacial drift of gravel, sand and boulders, a shale consisting of sand with a clay matrix, and finally limestone bedrock. The upper stratum of bed rock was identified by fossils and general appearance as belonging to the Gower stage of the Niagara series of Silurian rocks. This overlaid conformably rock of the Delaware stage of the same series. In the middle of the river, the Gower rock and nearly 50 feet of the Delaware rock had been entirely eroded. Great care was taken to ascertain the possible existence of subterranean pockets or overhanging cliffs in the rock. Only two of these pockets were found, however, both in the same boring, and these were only 1 and 6 inches in depth. Both were filled with sand, consisting of about equal parts of quartz and dolomite sand. Some of the borings were carried down 30 to 40 feet into the bed rock to determine the possible existence of these subterranean pockets.

"All the boulders encountered were such as could be easily broken with the chopping bit and no dynamite was found necessary. To determine the consistency of the shale, cores were taken out with saw-tooth bits working dry, showing perfectly the consistency of the material. The saw-tooth bit or the chopping bit working with the pump gave no idea of what this material was, and without the expedient of the dry core an excellent foundation would have been overlooked, and a foundation sought 30 feet lower.

"Borings in the limestone were made with a bortz bit when the water was still, and with the chopping bit, taking occasional cores with the saw-tooth bits. Fully 95 percent of the borings in the limestone were made with the bortz bit. The aggregate length of casing put down was 692 feet, and that of casing driven through hard material was 406.5 feet. The aggregate length of borings in shale was 86 feet, and in limestone, 226 feet. The cost was as follows: labor, $456.16; coal, $124.41; depreciation of bortz, estimated, $200.00; scow, $287.24; and depreciation in tools, pipe, etc., $200.00; total, $1267.81. The scow still has value which is somewhat uncertain. Omitting this credit, the cost of the work amounted to $1.83 per foot."

In exploring for the tower foundations of the Williamsburgh bridge, wash borings were first made, the pipe being driven to what appeared to be rock, and then a dynamite cartridge was exploded. If the pipe could not be driven farther after the explosion, it was at first assumed that bed rock was reached. Upon making diamond drill borings it was found that instead of 50 feet to bed rock on the New York side, the true depth varied from 46.1 to 68.3 feet; and on the Brooklyn side, instead of 75 to 80 feet, it varied from 80 to 104 feet. In nearly every case the wash borings had met a large boulder, and the charge of dynamite was not sufficient to break it. The diamond drilling was extended from 10 to 20 feet into the rock.

A summary of Experience in Diamond-drill Work on the Deep Waterways Survey with Statistics of Cost is published in Engineering News, vol. 50, page 83, July 23, 1903. The data relate to 25 holes of an average depth of 98.5 feet. The conditions under which the work was done are described and the special difficulties noted. The total carbon loss is 25 6/64 carats in drilling 1688 feet of rock, making the cost for diamonds 47.7 cents per foot, at $36.50 per carat. The rate for drilling in different kinds of rock, in feet per hour, is: quartzite 1.7; limestone, 2.5; sandstone, 3.0; and shale, 5.0. The distribution of time is as follows:

	Hours	Percent
Sinking casing	325	17
Drilling rock	753	41
Delays	386	20
Moving (38 times)	356	19
Holidays and storms	60	3
Total	1880	100

The delays due to moving were unusually high on account of frequent and long moves, as well as bad weather and delays in getting cars.

The total length of casing sunk was 552 feet, and of holes drilled in rock, 1910 feet. The general average depth sunk per day of 10 hours is, therefore, 13.1 feet when moving and other

delays are included, and 16.2 feet when the time for moving is excluded. While actually working, the rate of sinking the casing through sand, gravel, etc. was 17.0 feet, and of drilling in rock, 25.3 feet per 10-hour day. The analysis of cost is given including 13 items, the total cost per foot averaging $3.137.

ART. 181. CORE DRILLING WITHOUT DIAMONDS

The increasing demand for black diamonds for diamond drilling in mine prospecting and for other purposes led to such a rise in price as to exceed 10-fold the cost when the core drill was first practically developed. This naturally led to the invention of core drills without the use of diamonds. In one type of such drills, chilled-steel shot are made to travel under a hollow soft-steel bit which rotates and exerts a pressure on the shot at the same time, thereby causing them to mill away the rock. The bit is screwed to the core barrel and that is screwed to the hollow drill rods as in diamond drills. One side of the bit has a V-shaped or diagonal slot in it to aid the shot in working freely under the bit and to permit some of the water from the jet to escape without passing under the edge of the bit.

The shot varies in size from dust to particles as large as duck shot, the average being about 3/32 inch in diameter. This shot is fed automatically at a uniform rate during drilling.

Another special feature of one make of shot drill is the calyx or sludge receiver. It is formed by a tube surrounding the drill rods above the core barrel, in which are deposited the chips or sludge on account of the sudden decrease in velocity of the upward current of water. The cuttings thus received form a duplicate record of the strata penetrated. If sufficient water is used to bring the cuttings to the surface, its velocity is so great as to wash the shot away from under the bit.

Figure 181*a*[1] shows a drill of this type, the shell just above the point marked *I* being perforated to admit the sludge. To remove the core, small gravel is dropped into the pipe and this wedges between the core and inner wall of the bit near the bottom

[1] Courtesy Ingersoll-Rand Co.

when the drill is given a few turns. This breaks the core and permits lifting it.

The sizes of cores cut by shot drills are generally larger than those of diamond drills and range from 1½ to 20 inches. The largest cores are required for other purposes than foundation explorations. Cores 4 to 5 inches in diameter can be extracted as cheaply as those of 2 inches or less, while the rate of progress is as good or better than for the smaller cores. Except for the very hardest rocks, the shot drill is found to be more economical than the diamond drill.

The successful operation of the drill requires the proper regulation of the amount of water necessary to remove the cuttings without displacing the shot; and of the pressure to exert upon the bit to obtain the most effective cutting. The most serious difficulty is due to crevices in the rock in which the shot may be lost, requiring either some means of artificially sealing the openings, feeding the shot slowly but continuously by an expert operator or substituting a toothed bit to drill past the crevice. Some

FIG. 181*a*.—Shot Drill.

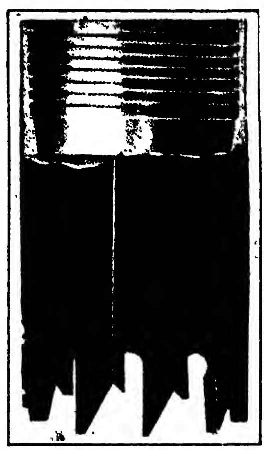


FIG. 181*b*.—Davis Cutter.

machines are arranged to embed the shot in the bit by churning instead of direct pressure.

In another type of drill without the use of diamonds, steel bits are used with different forms of teeth, and also operated by rotation. One of these, known as the Davis cutter (Fig. 181*b*), has long, tempered steel teeth with angles from 30 to 35 degrees, while between them are vertical grooves on the outer surface of the bit. Instead of grinding with a uniform motion, the long teeth chip away the rock by an action closely resembling that of a hammer and chisel. This cutter is used for the softer rocks, as well as for other material overlying the rock, instead of a chopping bit.

The ordinary churn or percussion well drill is sometimes used for subsurface explorations, being particularly valuable in boulder-strewn gravel. It will drill through rock, but only in the soft rock can cores be gotten. This drill is cheap to operate and rapid progress can be made.

The machine consists of an apparatus for raising and lowering, with a churning motion, a string of tools consisting of a hardened steel chopping bit screwed into a stem, which, in turn, is attached to a set of jars held by a rope. These jars are for jerking the bit loose when it sticks in the bottom of the hole.

Only a small quantity of water is required to make a creamy mass of the rock as it is broken. From time to time the hole is bailed out by a sand pump, consisting of an iron pipe with a valve on the bottom.

When a stratum is reached at which it is desired to take a core, a special core drill replaces the steel bit and stem. The bit of this core drill is similar to the Davis cutter and inside the drill rod there is a core barrel. This barrel is free to move up and down inside the drill rod and extends down 4 feet beyond the bottom of the bit. As the drilling stroke is only about 18 inches, the barrel is not lifted off the bottom by the motion of the drill. After drilling about 2 feet, the tools are drawn and along with them the core.

An excellent article, by ROBERT RIDGEWAY, on boring methods and machines, difficulties encountered and results of

explorations in various mixtures of glacial materials as well as rock, is entitled Subsurface Investigation on the Catskill Aqueduct, Board of Water Supply, in Engineering Record, vol. 57, pages 522 and 557, Apr. 18 and 25, 1908. See also Chap. IV on Borings and Subsurface Investigations in a volume on The Catskill Water Supply of New York City, by Lazarus White, New York, 1913.

Art. 182. Need of Subsurface Explorations

It is as essential to the proper design of foundations to determine accurately the local conditions under the surface of the ground or below the bed of a stream as to observe the controlling conditions of the stream itself, or to know the character of the superstructure and the magnitude and direction of all the external forces acting upon the substructure. To discover the character of the underlying strata and to find their respective depths below high and low water in the case of a bridge, or below the surface or the level of ground water in the case of a building, it is necessary to make excavations, or borings, and in some instances to drive test piles. In many locations conditions vary greatly within short distances.

Adequate exploration is often omitted because of the labor, time and cost. The cost of exploration, however, is frequently less than that otherwise required merely to revise the plans of the structures involved, without considering the unnecessary cost of the structures due to lack of information. There are abundant examples to prove that, where adequate exploration is omitted, it may result in the loss of the structure, or in greatly increased cost. In one instance a bridge pier was built upon a surface of hardpan in the river bottom. No examination was made on account of the swift current, which had a velocity of 5 miles per hour. Without warning the pier sank out of sight, causing the loss of the two adjacent spans of the bridge and of a number of human lives. Upon making an investigation afterward it was found that the hardpan was only a thin stratum overlying a deep layer of soft clay. In another

example a bridge abutment which was founded on 60-foot timber piles settled slowly until it reached a maximum of 3 feet, after an attempt to stop it by means of additional piles around the outside. Exploration proved the settlement to be due to a 10-foot layer of peat 35 feet below the surface, which was flowing apparently under the superimposed load. This experience emphasizes the statement made in Art. 38 that test piles alone may be insufficient. Experience has also shown that the cost of exploration may frequently save much larger sums in the annual cost of maintenance of structures like pile trestle bridges. In a certain project the chief engineer estimated that $100,000 was saved in the cost of construction by a thorough preliminary exploration of the ground.

Another important reason why adequate explorations should be made is that the owner ought to assume full responsibility for the local conditions and that the contractor should not be obliged to gamble on uncertainties relating thereto. The contractor should be asked to bid on guaranteed local conditions, with an increase or reduction in price for variations from these that may be discovered later. Occasionally, inadequate explorations may be made which are equally unfair to the contractor. For example, only four borings were made on a city block to be covered by an important building, one at each corner; but it was distinctly stated that these borings were furnished as general and not as specific information to the contractor, and that he must assume all chances as to the subsurface formations. The contract price for the foundations alone was $208,453.

An engineer of large experience made the following statement in 1910, based upon his own practice and his observations of ordinary conditions:

"I consider it very important and the expenditure well warranted first to determine definitely by test borings or drilling what the actual depth and character of the foundation is before any detailed plans are prepared. Such determinations enable the designer to design the structure as it should be built to meet the conditions, assuming that the test borings or drillings have been properly made, and such an order of pro-

cedure saves much time in the designing room by eliminating numerous changes in plans where unexpected conditions arise when the foundations are being excavated, and which is frequently the case. In nearly all cases there is a hurry to start the foundation masonry and frequently plans cannot be, or are not, properly modified to suit the conditions and meet the requirements of economy. Furthermore, with a proper knowledge of the foundations, the contractor is placed in possession of definite information, and with the plans properly designed once and for all, with possibly some minor modifications, the result is a large economy to the company paying for the work, which also eliminates questions of extra prices and frequently some arguments over changed conditions affecting unit prices."

In general, two sets of borings should be made for an important bridge crossing. In the first set a number of borings are located on the center line of the proposed location, to determine whether the site furnishes favorable conditions; and if so to make an approximate estimate of the most economical location of the piers and the length of spans. Sometimes government regulations for navigable streams or the influence of ice or flood conditions must also be considered. After the piers are located tentatively, additional borings should be made at the site of each pier. At least four borings, or one at each corner, are necessary, and several intermediate ones may be required unless the adjacent indications are nearly the same.

ART. 183. TESTS FOR BEARING CAPACITY

After an exploration of the different strata has been made, a test should be made at the surface on which the foundation is to rest to determine the bearing capacity of the ground. Figure 183*a* shows the appliances often used for this purpose, the hardpan being tested at the base of a pier to be built in an open well. The surface is scraped to a level plane by means of a straightedge. The platform is loaded with blocks of cast iron or other weights, and is transferred to the bearing plate at the foot of the post. The platform is held in proper position by wedges loosely placed against the sheeting, and the settlement is measured by taking readings on the steel tape at the top of the shaft. In one example of such a test, a load of 24,200 pounds per square

foot produced a settlement of $\frac{3}{16}$ inch in 44 hours. The working load adopted was 13,300 pounds per square foot.

When the platform is placed above the natural surface, it may be braced conveniently by extending the post above the height to be occupied by the loading, and into a loosely fitting collar which is held in position by four inclined shores; or four vertical timbers supported by shores may be placed so as to correspond to the four sheeting planks which take bearing from the wedges in Fig. 183*a*.

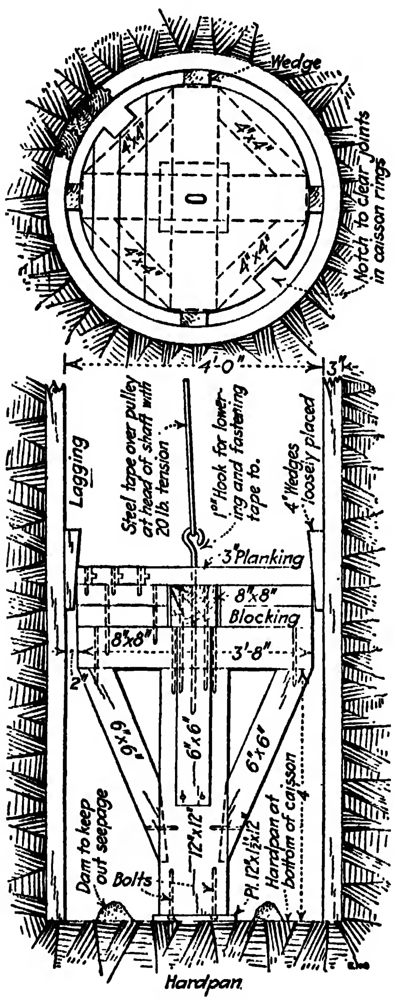


FIG. 183*a*.

To determine the bearing power of the sand on which some of the pneumatic caissons under the Municipal Building in New York City were to be founded, a 16-inch casing was driven, and cleaned out by means of a sand pump. Inside of the casing was placed a 10-inch pipe having a bearing disk 14 inches in diameter attached at the bottom and a balanced platform at the top to receive the load. Three tests were made at different depths. At a depth of 77 feet below the curb, loads of 15, 22.5 and 2 tons per square foot produced settlements of $\frac{37}{64}$, $1\frac{49}{64}$ and $2\frac{49}{64}$ inches, respectively. Later, one of the circular caissons was tested. Its base had an area of 90.8 square feet and was 72 feet below the curb. The

load was applied by increments of 1 ton per square foot every 24 hours. A load of 6 tons per square foot, which was that used in designing the caissons, caused a settlement of ½ inch without further increase during a rest of six days. After the load was increased to 10 tons per square foot, the total settlement was 15/16 inch without further increase.

On tests made in St. Paul on the bearing capacity of a mixture of coarse sand and comparatively fine gravel, 15 feet below the surface, the material was displaced and forced up around the base at a load of 6½ tons per square foot. In two days the settlement was 2½ inches. The bearing block was 2 feet square. A second test, carried out with a block 4 feet square, the remainder of the pit being covered with 2-inch planks weighted to 475 pounds per square foot, showed a settlement of only ¼ inch under a load of 8 tons per square foot.

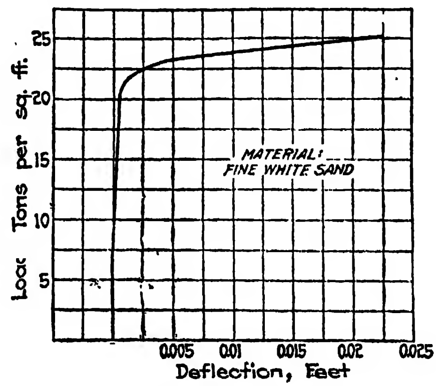


FIG. 183*b*.—Typical Load Settlement Diagram for Sand Foundation.

Tests made on fine sand at the bottom of a pneumatic caisson showed a bearing capacity of 20 tons per square foot with practically no settlement, while with loads greater than this the settlement became very pronounced. Figure 183*b* shows the load settlement diagram, which is a typical one for sand.

In Chicago, stiff blue clay, which a stout digger could shovel only with persistent effort, supported a load of 7½ tons per square foot with no settlement, while a load of 12 tons per square foot caused over 3 inches of movement in five days.

In the same city, hardpan, requiring considerable force to drive a miner's pick into it, supported a load of 15 tons per square foot with a settlement of 3/16 inch in 60 hours.

In another case a test made of thoroughly wet, fine sand showed a settlement of ⅛ inch under a load of 7½ tons per square foot, the load remaining on for one month.

For soil tests up to 10 tons per square foot the committee of the American Society of Civil Engineers, on the bearing value of soils, recommends, in the August, 1920, Proceedings,

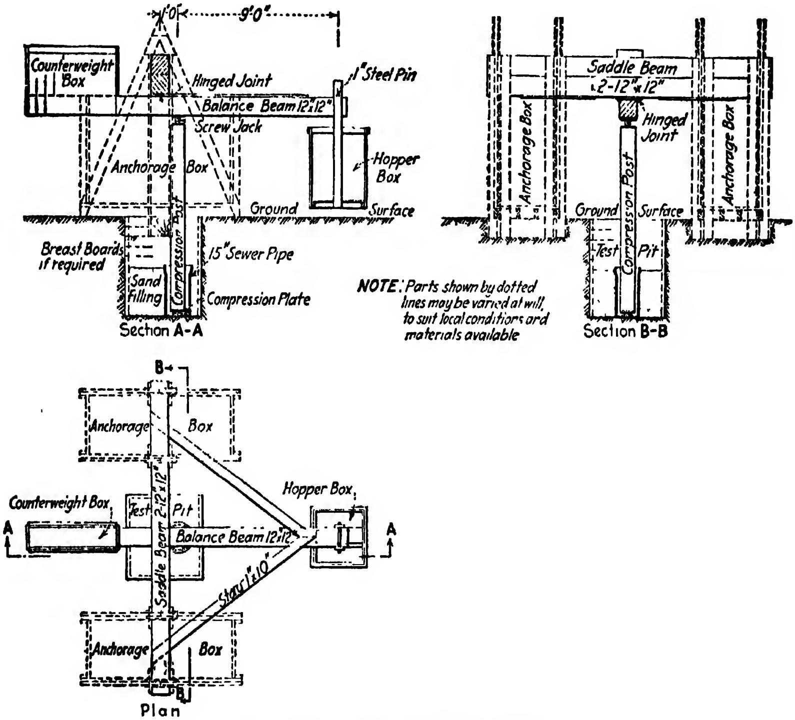


FIG. 183*c*.—Soil Testing Apparatus.

the machine shown in Figs. 183*c* and *d*. This machine has been used in a number of cases with success. Where the ground is wet, it has been suggested that in order to prevent a squeezing out of the soil between the edges of the plate and the tile which surrounds it, a strip of burlap be spread over the surface before placing the bearing plate, tile and back filling.

In making tests, the load should be applied at the uniform rate of 1000 pounds per hour for the period during which the

soil is being compacted and well into the period when displacement is taking place. For most soils, a sharp break in the compression curve appears at the point where displacement begins.

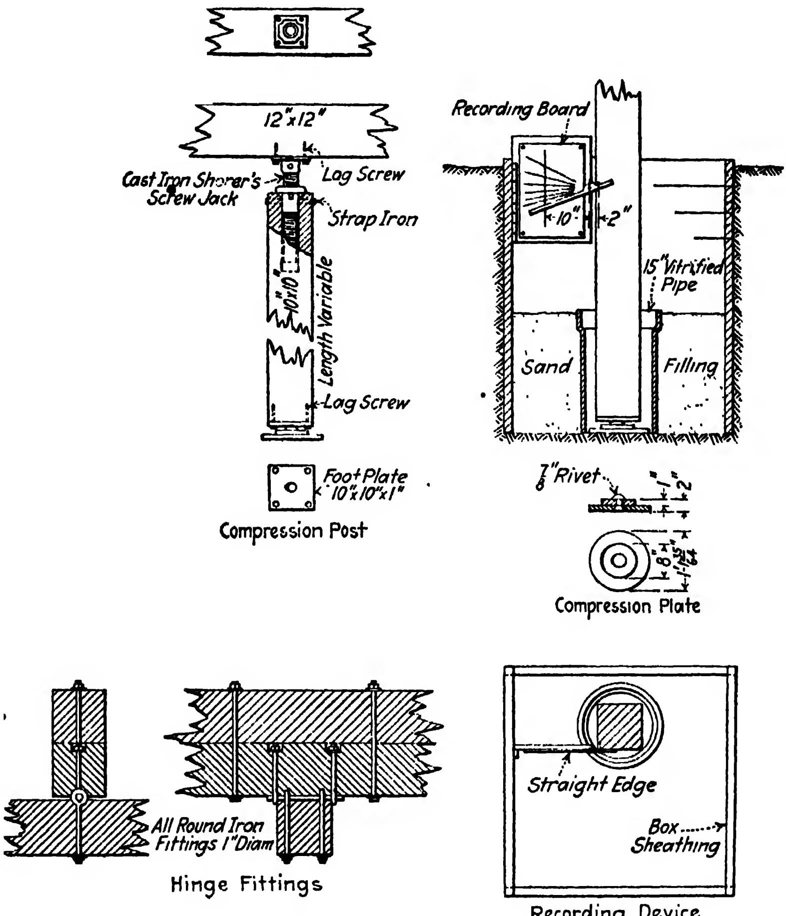


FIG. 183*d*.—Soil Testing Apparatus.

The safe bearing value of the soil is usually limited to one-half the load causing initial displacement.

Long-time tests are necesary for some soils, particularly clay. In one test on dense blue clay, where the bearing area was a circle 9⅓ feet in diameter, a load of 5485 pounds per square

foot caused practically no settlement for a period of eight days and then settlement started and continued intermittently for over a month. The total settlement was approximately 4 inches, three-quarters of which occurred with the load of 5485 pounds.

The loaded area has most frequently been taken at 1 square foot, but it is better to take a larger area like that specified in the next paragraph, and to subject it to a load not exceeding twice that of the proposed working load, than to overload a much smaller area. To get the best results, it is important to make the test on a surface as near the elevation of the proposed base of the foundation as possible, not at the surface of the ground, and with as little clearance as possible between the bearing disk or plate and the sides of the excavation or casing. It is also desirable to know whether any elasticity exists in the ground, and for this purpose the reduction in settlement should be measured, as the load is taken off in parts with short intervals of rest between. The unloading may be done more rapidly than the loading. Tests on beds of clay should extend over longer time intervals, depending upon their character.

An excellent graphic representation of the phenomena of tests for bearing power consists in laying off the time intervals as abscissas, the loads per square foot as positive ordinates, and the settlements as negative ordinates. The curve of settlement below the axis can thus be compared directly with the stepped load line above the axis, showing the effect of both increase in load and of intervals of rest after the successive increments of load.

The following extract is made from the regulations (1912) of the Building Department of the Borough of Manhattan:

"The soil shall be tested in one or more places as the conditions may determine or warrant, at the level at which it is proposed to place the bottom of the foundations of the structure. Each test shall be made so as to load the soil over an area of not less than 4 square feet in any one place. The accepted safe load shall not exceed two-thirds of the final test load. The loading of the soil shall proceed as follows: (*a*) The load per square foot which it is proper to impose upon the soil shall be first applied and allowed to remain for at least 48 hours undisturbed, measurements or readings being take once each 24 hours or oftener to determine the settle-

ment, if any. (*b*) After the expiration of the 48 hours the additional 50 percent excess load shall be applied and the total load allowed to remain undisturbed for a period of at least six days, careful measurements and readings being taken once in 24 hours, or oftener, in order to determine the settlement. The test shall not be considered satisfactory or the result acceptable unless the proposed safe load shows no appreciable settlement for at least two days and the total test load shows no settlement for at least four days."

ART. 184. VALUES OF BEARING CAPACITY

No definite values can be given in general to the safe loads on foundation beds, since it is impossible to classify the various kinds of earth accurately. Unless the bearing capacity of the material at a given site is already known it should be determined by direct experiment. Too little attention has been given to this subject and but small additions have been made in recent years to any real knowledge of the material on which the foundations of structures rest. For preliminary estimates and some other purposes, limiting values of bearing capacity may be employed for several classes of material.

In his General Specifications for Structural Work of Buildings, C. C. SCHNEIDER recommends that the pressures on foundations are not to exceed the following values in tons per square foot: soft clay, 1; ordinary clay and dry sand mixed with clay, 2; dry sand and dry clay, 3; hard clay and firm, coarse sand, 4; firm, coarse sand and gravel, 6. Other experienced engineers have characterized these values as needlessly conservative, but it was claimed in reply that conservative values were adopted because the effect of settlement in the foundation of a building is more injurious than in a bridge pier. The same specifications also contain a table giving the bearing capacity for different kinds of ground as prescribed in the building codes of a number of American cities.

The Building Code recommended by the National Board of Fire Underwriters gives the following limiting values, also expressed in tons per square foot: soft clay, 1; clay and sand together, wet and springy, 2; loam, clay and fine sand, firm and dry, 3; very firm, coarse sand, stiff gravel or hard clay, 4.

In H. B. SEAMAN'S Specifications for Bridges and Subways, the allowable static pressures are given as follows, in tons per square foot: silt, 1; moist clay, 2; clean sand or dry clay, 4; coarse sand or gravel, 6; hardpan or compacted gravel, 10; sound ledge rock, 60. It is stated, however, that these pressures are for the most favorable conditions, and that for questionable material they should be reduced 50 per cent. In deep foundations, friction and buoyancy may be allowed for in computation.

The following paragraph is quoted from J. E. GREINER'S General Specifications for Bridges, Part III:

"85. When foundations are subjected to the loads and forces specified in paragraph 84, the maximum permissible pressure per square foot on any part of the surface of the supporting strata, when in thick beds, shall be as follows, but it is advisable to use a considerably less pressure unless absolutely certain as to the character of the bottom. Firm rock, 30; dry coarse gravel and sand, well cemented, 5; hard, dry, compact sand, 4; hard, dry clay, 3 tons."

The following allowable loads, expressed in tons per square foot, were adopted Oct. 11, 1905, by the Harriman Lines: alluvial, adobe soil, 0.5; clean, dry sand, 2; compact sand, cemented, 4; gravel and sand, cemented, 8; moist, soft clay, 1; dry clay in thick beds, 4; soft bed rock, 5; hard bed rock, 20; hardest bed rock with no seams, 200.

The following are values based on the average of 40 building codes of the larger cities of this country:

MATERIAL	ALLOWABLE BEARING, TONS PER SQUARE FOOT
Quicksand and alluvial soil	0.5
Soft clay	1
Moderately dry clay, fine sand	2
Firm and dry loam or clay	3
Compact, coarse sand or stiff gravel	4
Coarse gravel	6
Gravel and sand, well cemented	8
Good hardpan or hard shale	10
Very hard, native bed rock	20
Rock under caisson	25

The most elaborate collection of data on unit pressures adopted for stable structures on the material upon which they are founded is contained in a volume entitled Allowable Pressures on Deep Foundations, by ELMER L. CORTHELL. The data relate to 178 works, and an analysis of some of them gives the following results:

Examples	Material	Range of pressure, pounds	Average, pounds
10	Fine sand..............	4,500–11,600	9,000
33	Coarse sand and gravel .	4,800–15,500	10,200
10	Sand and clay........	5,000–17,000	9,800
7	Alluvium and silt.......	3,000–12,400	5,800
16	Hard clay........	4,000–16,000	10,160
5	Hard pan.......	6,000–24,000	17,400

"These cases show no settlement. The range is considerable and no doubt in the case of the minimum pressure a much larger weight could have been imposed on the material without producing settlement. For a safe rule, therefore, the average is low and a safe pressure upon the material would lie somewhere between the average and the maximum pressure." The same volume contains the pressure for instances in which notable settlement took place, as well as values of frictional resistance for cylinder and masonry piers.

The bearing capacity of the ground depends not only upon its character or composition, but also on the amount of water which it contains or is liable to receive, and the degree to which it is confined to its location. Sand can sustain very heavy loads with but slight or negligible compression. When it directly overlays rock or some other thick stratum of hard material and is securely confined, or is artificially protected against the possibility of lateral displacement, it forms a satisfactory foundation bed and will safely carry heavy loads.

The supporting power of clay is very variable and depends in a large measure upon its variety and upon its degree of saturation with moisture. The clays vary considerably in their chemical constituents, which in turn affect the amount of mois-

ture which they can absorb. Certain deposits are known to be compact and hard and have a high supporting power, while others are plastic and easily compressed. The chief characteristic which renders clay more or less unstable as a foundation material is its property of retaining water which is once admitted, and its tendency to soften gradually as the amount of water increases. In plastic clay and other soft material the depth of foundation should enter as a factor in determining the allowable pressure. In other words, the so-called buoyancy of the foundation material is a function of the depth of "displacement" of the building or other structure. The point of bearing must be carried below the possibility of upward reaction alongside. This principle has sometimes found expression in a practical rule that "in compressible ground the depth of a foundation ought not to be less than one-fourth of the intended height of the building above ground; that is, for a shaft of 200 feet, the foundation should be made secure to a depth of 50 feet by piling, or by well-sinking and concrete. Masses of concrete, brick or stone, placed upon a compressible substratum, however cramped or bound, may prove unsafe. Solidity for a considerable depth alone can be relied upon. Mere enlargement of a base may not in itself be sufficient." It must also be remembered that large areas of compressible ground will not continuously support as large a unit load as a smaller area for a short time.

When clay is mixed with other materials, like coarse sand and gravel, its supporting power is considerably increased, being greater in proportion as the other materials are in excess up to the point of forming a cemented mass, in which the clay is just sufficient in quantity to act as a cement in binding the other materials together. In this condition the clay is often found in an indurated state, and the hardness of the mixture, commonly called hardpan, is proverbial.

CHAPTER XVIII

PNEUMATIC-CAISSON PRACTICE[1]

ART. 185. HISTORICAL NOTES

In 1852 an attempt was made in the Pedee River in North Carolina to use the ingenious vacuum method, invented by POTTS, to place the foundations for a bridge. His plan consisted in exhausting the air from cylinders 6 feet in diameter, thereby causing the atmosphere to exert a pressure of 14.7 pounds per square inch, or a total pressure of about 30 tons, which it was thought would force each cylinder through a depth of 25 feet of sand. Unfortunately, the attempt proved unsuccessful, since no allowance had been made for the presence of logs which were encountered under the cutting edge.

Accordingly, the opposite method was tried, or that of pumping additional air into the cylinders, thus introducing in America what was designated as the plenum pneumatic method. By this method the air is compressed sufficiently to balance the water pressure, thus keeping the water out of the working chamber. As a cubic foot of fresh water weighs about 62½ pounds, the pressure per square inch at a depth of 1 foot is 0.434 pounds and at a depth of 100 feet, 43.4 pounds.

The second set of caissons to be sunk in America were those of the Third Avenue bridge over Harlem River, New York, about 1860, the third being those for the famous steel-arch bridge at St. Louis, begun in 1868. The earliest patent for a compressed-air shaft was granted to THOMAS COCHRANE in 1830, while the first application was made in the river Seine, at Loire, by M. TRIGER in 1839. Compressed air had been used in diving bells, however, for many centuries, although the first

[1] THOMSON, T. KENNARD, C. E., D. Sc., Consulting Engineer, 50 Church St., New York City.

record of using a pump to compress the air in a diving bell is that of SMEATON, a noted English engineer, for repairing the foundations of a bridge over the river Tyne, at Hexham, in 1778.

ART. 186. RESULTS OF EVOLUTION

The first and second sets of caissons referred to in Art. 185 were iron cylinders only 4 to 6 feet in diameter. The first large caissons were built for the deep foundations of the Eads bridge at St. Louis and the Brooklyn bridge in New York. They were of very massive timber construction on account of the stone masonry being placed directly upon the deck. The heavy roof or deck was also used in the early days of concrete by some engineers, but in the decade following 1880 the thickness was reduced to less than 3 feet for bridge piers in some of the large rivers. In the Hartford stone-arch bridge, the largest caissons were 46 by 131 feet in size and had a timber deck only 4 feet thick. As these caissons had no bulkheads or diaphragms, they contained the largest undivided air chambers ever used.

Massive framed timber cribs with solid walls and heavy bracing were also used above the caisson deck, but the walls were reduced in time to 3-inch planking with sufficient timber bracing to resist the water pressure and possible bumps from boats, etc.

The first building ever founded on pneumatic caissons was the Manhattan Life Building at 66 Broadway, New York, in 1893. The caissons were of heavy steel construction, about 9 feet high with a 7-foot working chamber. It was intended to build the brick piers directly on the caissons as they sank, but the friction of the ground against the brick wall was too great for the green mortar and forced open the joints between the brick. This method was then replaced by building steel cofferdams above the caisson and filling them with concrete. The steel, in turn, was supplanted by wood, on more recent work.

Caissons and cofferdams built of steel with rectangular horizontal sections were used for many years, but have been

abandoned in America, while those with circular sections are restricted to small diameters. Except for the small sizes, it is always cheaper to use wood and concrete; and in many cases it is cheaper to use wood only for forms when all the concrete can be placed before sinking starts. This has often been done successfully up to heights a little over 30 feet above the cutting edge. This method is not economical, however, when the depth of sinking is so large as to require two or three "build-ups" of concrete during which the sinking must be interrupted.

To stop sinking temporarily is bad, since it requires continual pumping of air during the interval, thus increasing the overhead charges, and also allows the ground to cake against the sides, thus greatly increasing the friction. In one example, the movable derrick was shifted to another caisson instead of waiting several days for the "build-up," as the adding of concrete is called, and it was 60 days before sinking was resumed on this caisson. During this period, compressed air was pumped into the working chamber to keep the ground from caving in, although it would have been cheaper to flood the caisson for that purpose. It is much cheaper in such cases to use a cofferdam of 2-inch plank and to work continuously until the penetration is completed.

To sum up the results of evolution, it is found to be the best practice to use steel for small circular caissons, say from 30 inches to 6 feet in diameter; in larger sizes to use mass concrete with 2-inch timber sides, with reinforcement around the shafts, working chamber and sides, where it is necessary to keep joints from opening. It is believed to be better to leave on the thin timber sides, since it avoids unnecessary delays in sinking, reduces the skin friction as well as the liability of rupturing the concrete due to the friction, and makes it easier to keep the caissons plumb and in position.

Caissons with a timber roof and sides for the working chamber and with timber cofferdam on top are economical only for deep sinking in harbors or rivers, where buoyancy is an advantage. The writer has sunk caissons in water 60 feet deep where the cutting edge had to penetrate 30 feet below the bottom of the

river. The top of the concrete had to be kept about 25 feet below the water surface in the river, requiring great care in bracing the timber cofferdam and in guarding against concrete buckets knocking out braces, as well as injury from passing barges, derrick boats, etc. If no timber had been used in constructing the caisson, the distance from the water surface to the concrete being deposited would have been much greater.

ART. 187. CONSTRUCTION OF CAISSONS

For constructing caissons from 30 to 36 inches in diameter, such as are used chiefly for underpinning purposes, cast iron is preferable, with a thickness of 1¼ to 1½ inches. The sections should be about 5 feet long with substantial flanges to bolt them together, shorter sections being ordered to make up the requisite total length, which varies in each case.[1] Steel plates ⅜ inch thick have been used, but in some cases were badly twisted or warped while being jacked down, thus greatly reducing if not destroying the value of the cylinders as columns. Cast iron is much less liable to rust than steel.

When caissons are sunk in the open for new buildings, it rarely pays to use a smaller diameter than 6 feet; the working chamber then consists of a steel shell about ⅜ inch thick, with a steel-angle ring above the cutting edge and one or two more rings of, say, 3½- by 3½- by ½-inch angles between the cutting edge and the deck. If the steel cofferdam above a caisson of this diameter is omitted and removable forms used, the ring of concrete between the inside shaft and outside surface is only from 1 to 1½ feet in thickness, and unless the concrete is heavily reinforced, both vertically and horizontally, the concrete is sure to crack, as has often been proved by experience. The size and the spacing of the reinforcing bars depend on the depth of penetration, and in these circular caissons, 6 feet or over in diameter, it is often more economical to use wooden sides from the cutting edge to the top.

[1] See illustrations in an article on foundations of the New Mutual Life Insurance Building, New York City, in Engineering News, vol. 45, page 221, Mar. 28, 1901.

In the larger caissons, whether of wood or concrete, the first detail of construction, and on which there is still the widest difference of opinion, is the cutting edge. Naturally, everyone wants the bottom of the caisson to be as thin as possible, so that the sand-hogs can remove the material under the cutting edge with the least difficulty. Most superintendents demand a "knife cutting edge," which is a mistaken policy, for in fine sand, where such a knife edge works like a charm, it is not needed, whereas in other material where it seems to be needed it cannot be made strong enough to stand the terrific pressure. It is sure to be buckled and then becomes worse than no cutting edge at all, often causing considerable delay, while it is being removed.

The best form of cutting edge for large timber or concrete caissons consists of an 8-inch channel laid flat with its back down, or a 12-inch oak block sized down to 8 inches on the bottom. The 8-inch channel cutting edge was first designed by the writer for Arthur McMullen & Co., in 1901, and it has been used by them frequently. It is the most economical form.

The side walls of the working chamber require great care and judgment in their design. Theoretically, if the caisson is plumb, and the air pressure just balances the outside pressure, there is no pressure on the sides except that due to the load on the top, including the weight of concrete placed. But caissons are very rarely absolutely plumb, and they often get badly warped as well as inclined, due to more obstruction on one side than on the other, causing the sides frequently to buckle, sometimes to collapse or to break away from the roof. In one instance, where the last kind of accident occurred, the cutting edge was in sand about 20 feet above rock, and had to be left there, the balance of the excavation being made by a vertical tunnel method. In another example, the side walls of the working chamber made of 3/8- and 1/2-inch steel plates, braced firmly every 2 1/2 feet, have been observed to buckle at least 2 inches under the outside pressure.

Timber side walls are most readily braced and repaired, but must be firmly attached to the structure above to prevent

breaking away. In another instance, a reinforced-concrete caisson landed hard on one side, and in consequence the steel rods from the cutting edge up were badly buckled, forcing the concrete side walls into the working chamber, leaving the outside timber bare, which, fortunately, had been left in place. Curiously, the thick reinforced-concrete wall was destroyed, while the 3-inch plank remained.

In small caissons the cutting edge and sides should be made strong enough to withstand the pressure without cross-bracing, but in large caissons it is customary to use substantial struts every 10 or 15 feet, and most designers use solid-timber bulkheads in long or wide caissons, making an air chamber about 20 feet wide. However, as every bulkhead makes two more cutting edges to work under, and since this part of the excavation is the most expensive, the writer has preferred to omit the bulkheads and has done so successfully up to a width of 46 feet and a length of 131 feet, at Hartford, Conn. The size and spacing of this bracing must be governed by experience. The caissons at Hartford are typical of others used successfully at Pittsburgh, Mingo Junction, Havre de Grace and Pierre.

The earliest wooden caissons had much heavier wooden decks than were needed to act as supports for the stone masonry. Steel caissons also had heavy beam deck construction, even when concrete was supported. It was long thought necessary to have a timber or steel deck to secure an air-tight job, but experience has shown that, by first placing a layer of mortar, it is easier to secure air-tightness with concrete than with wood or steel. Upon reflection, it was seen that under concrete a deck of timber or steel was needed only as a temporary form, except when the concrete was shallow and then it could be reinforced.

There are three good reasons for omitting the permanent wooden deck: First, it is more compressible than concrete and is liable to loads sufficient to compress it; second, concrete is cheaper than wood; and, third, danger of injury from the Teredo wherever it exists. On the other hand, it is often more economical, of time and money, to leave a thin timber shell, say 2 inches

thick, under the roof. By not carrying this to the sides, it may be made inaccessible to the Teredo. The old theory that the Teredo will only start work near the water surface is erroneous according to the observations of the writer, who examined piles driven two years previously for a bridge at Fall River, Mass., and which were cut off 40 or 50 feet below the surface, and carried a timber grillage 4 feet thick on which the granite masonry pier was built. The piles were eaten through, allowing one end of the pier to drop 2 feet. When several pile heads were cut off, brought to the surface and cut open, live Teredo and Limnoria were discovered, although the location was within 200 feet of the mouth of a sewer. Whether the Teredo, due to its objection to crossing joints, entering beyond a certain distance, etc., would destroy the deck of a large wooden caisson, is an open question, but the danger is great enough to rule out timber in the future whenever possible, at least in Teredo-infested waters.

There are some accidents or errors of judgment against which the designer is powerless. For instance, one of the best superintendents in the country put too large a charge of dynamite outside of the cutting edge to break up the rock and blew out the end of the caisson. Besides making it a total wreck, the jar combined with an extra high tide broke the bond between the concrete and bed rock in an adjoining caisson, about 200 feet away, and lifted the caisson enough to require its removal and rebuilding, although the air chamber had been filled with concrete and the caisson work completed. Numerous cases have occurred where the deck has been badly warped by allowing one corner to land on a harder substance than the rest of the cutting edge, or by not having the bed dredged to a uniform depth before placing the caisson. One of the Quebec bridge caissons was thus injured.

ART. 188. CALKING, SHAFTS AND LIGHTING

One of the advantages of concrete caissons is the absence of joints to be calked. In steel caissons the joints must be calked with a regular calking tool, while in wooden caissons

every joint must be tightly packed with oakum and often covered with a coat of pitch also, in spite of which there is usually a considerable loss of air.

In calking with oakum, one man can cover about 180 feet of joint in a day, going over the work twice but using only a single line of oakum. In wooden or steel caissons, it is very hard to get a water- and air-tight job, even with the best calking and flushing the deck with mortar before placing the concrete. No one likes to see the air bubbling up through the freshly made concrete, much less to see it form a water-spout several feet above the top of the concrete. In New York, the escape of air from below the cutting edge or elsewhere has been observed in buildings from 100 to 200 feet away.

The preceding statements about calking the side walls and deck of the working chamber apply even in a higher degree to the cofferdam above the caisson, especially as the top of the concrete may be many feet below the water surface. A small leak requires pumping and this is the greatest enemy of concrete.

Small caissons under 4 or 5 feet in diameter are practically all shaft, but in larger sizes, one or more inner shafts are used to take men and material in and out of the caisson. For sizes up to 12 or 15 feet in length, it is customary to have only one shaft for both men and material. Sometimes this is simply a material shaft, 3 feet in diameter, with a ladder set in the side, while at other times a combined shaft is used, an oblong affair divided into two compartments, one with a ladder for men and the other for taking material in and out.

Whenever the size of the caisson permits, there should be two or more shafts to avoid unnecessary danger to the men, and also to facilitate filling the air chamber. Temporary failure of the lock to work, or the jamming of a bucket in the shaft, has often held the men as prisoners for hours, sometimes more than twelve, and sometimes with serious loss of life. On the other hand, two shafts are more expensive than one, even allowing for the extra handling of material, and contractors do not like to incur the extra expenditure of time and money.

But several shafts do not always insure safety. Some years ago, in the Passaic River, one of the best foremen failed to fasten the bucket properly to the hoisting rope, and hence the bucket dropped in the lock, forced the bottom door open while the top was also open, thus allowing all the compressed air to escape and drowning nearly all the men in the working chamber, including the foreman himself.

If the shaft is made of steel and not buried in concrete, it should be ⅜ inch thick, and properly fastened so that neither shaft nor lock can be blown off, as has sometimes occurred. If the steel shaft is buried in the concrete, it may be built of ¼-inch metal, provided it is designed so that it cannot be blown off. At first, it was customary to use heavy steel shafts which were left in the concrete, but this proved so expensive that the plan was modified to leave only the bottom section buried in the shaft, its length not exceeding 8 to 10 feet and sometimes only 18 inches. The rest of the shaft was protected from the concrete by a timber box surrounding it. But as the concrete often leaked through the box and set around the shaft, preventing its removal, and causing considerable loss to the contractor, two other methods were adopted later. In the first plan, a heavy cast-iron collapsible shaft is used, which is removed after the air chamber is filled. In the second plan, timber forms with rungs for a ladder are used in the shaft, except for the upper section near the lock, especial care being taken to provide an adequate connection of the steel shaft to the concrete. A number of fatal accidents have occurred, in which the lock was blown from the caisson.

When candles were employed for lighting, there was constant danger of fire. A fire started in the joints of a timber deck and fanned by compressed air is very difficult to put out, even by flooding the working chamber. More recently, upon hoisting a bale of oakum from the 4-foot joint well between two caissons, a candle in the lock was knocked over, the rope set on fire and the blazing oakum dropped on the men below. Two men were burned to death and several others seriously injured.

All caissons should be lighted by electricity whenever possible, even the small joint caissons between the main ones, as the accident just cited indicates. Apart from the danger of fire, the old tallow candle was never satisfactory, for, as the rate of combustion is greater in compressed air, the lungs of the workmen are so filled with soot that many days are required to get rid of it. Electric-light wires are generally carried down the shaft and occasionally a bare wire will charge the iron ladder, giving an unwelcome shock to the men using it.

The pipes, 3 or 4 inches in diameter, which convey the compressed air to the working chamber, as well as the gas pipe for whistling or signaling to the men outside, were formerly left in the concrete, but in later practice they are sometimes placed in a pocket next to the shaft, so that they can be removed and used again. The same arrangement is applied to the pipes, about 5 inches in diameter, which are used to blow out material when that method is suitable. These details and many others can be designed only by one who has worked in a caisson and who is familiar with methods of operation.

ART. 189. METHODS OF LAUNCHING

There are practically five methods of getting a caisson into the water: First, when built on shore, it is skidded into the water on launching ways; second, when built in a pontoon, the pontoon is taken away underneath; third, when built on a boat or wharf, the caisson is lifted by derricks and placed in the water; fourth, when the cutting edge is supported from a temporary platform on piles or boats, and is then lowered by long screw rods, or block and tackle, etc., to a firm bottom; and, fifth, constructing an artificial island at the caisson site.

When built on shore and skidded into the water, no more work is done before launching than is necessary; the bottom of all shafts should be closed temporarily and a small amount of grout or concrete placed on the deck to make it water-tight. A caisson of ordinary size and timber construction will draw about 8 to 10 feet of water when floated and must have enough cofferdam to prevent flooding. A failure to provide skids or

runways of ample strength has resulted in several breakdowns before launching, with a loss of thousands of dollars and considerable valuable time.

The method of building on a pontoon is very satisfactory, especially when a number of caissons can be built on the same pontoon. A pontoon is a flat-bottomed boat with vertical sides, leaving a clear space of about 5 feet in which to work all around the caisson. The bottom is constructed of 12- by 12-inch timbers spaced 2 to 3 feet apart, to which 4-inch planks are spiked underneath. The sides are 6 or 7 feet high or sufficient to prevent any danger of flooding during construction. Both the bottom and sides are thoroughly calked with oakum.

These pontoons are made of two or more parts bolted together in the middle and so arranged that after the caisson has been built to a height of 14 to 20 feet above the cutting edge, calked, with shafts, etc., in place and properly connected to the deck, the bolts connecting the two halves can be removed. Stone or gravel is then placed on the center of the pontoon, and when everything is ready the valves are opened, allowing the pontoon to fill with water until the caisson floats. Sometimes the arrangement is so perfect that the minute the caisson floats the two halves of the pontoon shoot from under and the launching is completed. It is often necessary, however, to attach tugs to pull the sections apart; or, by means of struts attached to the caisson and by block and tackle, the pontoon sections are pushed apart. At one time a superintendent forgot to sink the pontoon first in order to relieve it of the weight of the caisson, and upon trying to pull away the pontoon sections he succeeded merely in letting in water enough to freeze the caisson to the pontoon; accordingly, it took two weeks instead of about three minutes to launch it. At Hartford, where seven caissons were 23 feet wide and two were 46 feet wide, the pontoon was built for the smaller size and additional sections were added for the larger size.

Building on boats or shore and lifting the caissons bodily into the water depends upon local conditions, and applies to the

smaller caissons. The fourth method is in some cases the only one which can be adopted economically. For example, at Pierre, S. D., the bed of the Missouri River consisted of very fine silt and the water was too shallow to float a caisson; if a channel 10 feet deep were dredged out, it would fill up before the caisson could be towed into place. Piles were therefore driven to form the supports for a platform around the site of each caisson and about 32 rods were suspended from these platforms in such a way that the cutting edge could be built upon their hooks at the bottom. After the caisson was built up about 14 feet above the cutting edge, it was lowered by simultaneously turning the nuts on the 32 rods until the cutting edge came to rest on the bottom. The rods were then disconnected for use on the next caisson while building up the cofferdam and concreting were continued. It took from 10 to 12 hours to lower a caisson.

ART. 190. PLACING AND SINKING

Before launching a caisson and lowering it to position, the site must be prepared by excavating the higher spots to a level surface. If the low spots are filled, they are not so firm as the other material and thus cause trouble. Leveling the site properly is especially important in a swift current. When possible, guide piles are driven on each side and sometimes clusters of piles are driven up- and downstream to which lines are attached to hold the caisson in position until it is sunk deep enough to be safe. These guide frames support working platforms, form parts of supports for derricks, unless they are mounted on boats, act as wharves for boats of stone, sand or cement, and for the "sand-hogs" boat. In some cases, it is advantageous to build a temporary island of gravel, sand, etc., on which to build the caissons in position, looking out for the danger of floods, since some rivers rise enough in 24 hours to wash away such an island.

After the caisson has reached the proper position, the shafts are built up and concreting started until the cutting edge has

penetrated far enough into the ground to make it safe to put on air and send down the sand hogs. In the Mohawk River, caissons were started on artificial islands, while in the Susquehanna River, at Havre de Grace, they had to be sunk through 60 feet of water. At the former locality, the concrete was well above the water surface from the start of sinking, while at the latter the concrete had to be kept 25 to 30 feet below the surface until the bottom was reached.

The ideal condition during sinking is to have just weight enough to keep the caisson moving gradually and continuously, with the cutting edge a few inches below the excavation in the working chamber, until the final position is reached, but it is difficult to secure this condition. If a caisson is too heavy, it is liable to break the side friction and fill the air-chamber with sand, and if it is not heavy enough, as frequently happens, it is necessary to lower the air pressure to start the movement, thus giving jerky sinking.

In caissons for city buildings, it is a common occurrence to see the excavation carried from 1 to 2 feet below the cutting edge, and then to have the air pressure lowered for a few seconds, the friction being suddenly overcome, and the caisson sunk 2 feet or more. Sometimes, however, hundreds of tons of pig-iron or cast-iron blocks are piled on top to assist the operation of sinking. The caissons for the Zinn Building in New York[1] first penetrated made ground and the Hudson River silt. On lowering the air pressure, the friction was suddenly overcome and the caisson sank until sand and mud filled the air chamber, resulting in the loss of a shift of eight hours. This accident occurred 15 times at that site, sometimes without lowering the pressure; a record of misfortune which has never been equaled. Fortunately, the men were in the shaft when it happened the first time and were on the watch afterward, so none were hurt.

In sinking the first caisson for the Municipal Building in New York, when the penetration was nearly 100 feet below ground-water level, the air pipe broke and within 15 minutes sand filled the working chamber and extended 16 feet up into

[1] See Canadian Engineer, Feb. 22, 1912.

the shaft, while the water had risen 42 feet above the cutting edge. The men were on the way up the shaft when the connection broke and the water followed the feet of the last man nearly as fast as he could climb. If plenty of weight in the form of iron blocks can be obtained without too much cost, it is better to use it in sinking, for reducing the air pressure or using a water-jet is almost sure to increase the friction for the next drop, with exasperating results.

It is essential to have sufficient outside bracing to keep the caisson in line until the penetration reaches 25 to 30 feet. If it gets far out of line before that, it is almost impossible to plumb it again, and is likely to get out of line still more as it sinks. If it is nearly plumb at that depth, there is rarely much trouble at greater depths. The more it is out of plumb the more is the caisson apt to be warped, greatly increasing the frictional resistance. Very few caissons, however, are less than 6 inches out of plumb, or out of line, and a greater allowance than this should always be made in designing foundations.

It is useless to specify that no caission will be accepted, if it is more than 6 inches out of plumb or position, if sunk to any considerable depth. It would be a radical operation to remove a caisson sunk from 40 to 90 feet and start over again, for generally the loss of time to the owner would be sufficient to prevent enforcing such a provision against the contractor.

Pneumatic caissons are used only where water is encountered, and where the volume of water is too great to permit pumping in an open cofferdam, or where such an operation would endanger adjoining structures by drawing the water and sand from under them and thus allowing settlement. The air pressure must be just sufficient to keep the water from flowing in and bringing the sand with it. Even when much care is employed with compressed air, trouble on this account occurs frequently and sometimes at a considerable distance away. For instance, while sinking the foundations for Liberty Tower—now the Sinclair Building—in New York City, a certain quantity of water and sand must have escaped from under the Chamber of Commerce Building on the opposite side of the

street, causing the interior columns to settle considerably while the outside walls were apparently not disturbed.

ART. 191. EXCAVATION AND SEALING

Two methods are in use for removing material from the working chamber: First, by buckets and derricks; and, second, by blowing it out. In the first method, the sand-hogs shovel or lift the material into buckets, which hold about ⅛ cubic yard. The bucket is attached to a cable and hoisted into the lock at the top of the shaft, the bottom door is closed and the top door opened, thus allowing the bucket to be swung clear of the lock, emptied and returned to the caisson for another load without having been disconnected from the cable. Occasionally, the lock has no top door but one at the side, in which case the bucket is dumped while still in the lock. Such a lock works better with sandy ground than with sticky clay. The makers claim that it is more economical than other locks, especially for small sizes. The best of these is the Mattson Lock, which has often been used to advantage.

When conditions permit the use of the blow method, this is the cheapest. It requires a 4- or 5-inch cast-iron pipe from the surface to the deck, from which is extended a flexible hose with a valve near the lower end. Above the surface the pipe must have a bend or elbow to direct the material away from the caisson. In operation, the material is shoveled or washed into a pile at the end of the hose, and the valve opened to let the compressed air carry it out. The material can be removed much faster than it can be shoveled into a pile, or than the concreting can be added at the top. A large volume of air escapes, while gravel and even fair-sized stones go out with such terrific velocity that a cast-iron elbow 2 inches thick is worn through in an hour. In one instance, the windows of a tug 200 feet away were broken. Accordingly, the hardest manganese steel is used for these elbows. While waiting for a new elbow, the expedient has been adopted of fastening a 12-inch block of wood to the old elbow.

The most important part of pneumatic-caisson work is in sealing the air chamber, or filling the space between the bottom of the excavation and the deck of the caisson with concrete. By the old method, the concrete was spread on the bottom until it extended a foot or two above the cutting edge, and then it was benched up around the sides, using boards for bulkheads if necessary, until the concrete was 3 or 4 inches below the deck. The remaining space was filled by ramming into it a fairly dry mortar. This method was very expensive and unsatisfactory, for the concrete had to be fairly dry to stand benching. Dry concrete should never be used in compressed air, since the moisture is absorbed so rapidly. The writer has examined old work and found the concrete, which had been placed in this manner, in a very poor condition. He has also taken out concrete, which was mixed very wet, from the bottom of a caisson and found it to be exceptionally good. In another method, the filling was continued either by bucket or concrete chutes until the wet concrete reached the roof. Actual measurements have shown a space of ½ to ¾ inch between the concrete and the deck, due to the shrinking of the concrete while setting.

The writer's present practice is as follows: The roof is sloped as much as possible and air vents 1 or 2 inches in diameter are placed as far as possible from the shaft used for the concrete. The air chamber is filled with very wet concrete to within 10 or 12 inches of the roof. Meanwhile, the air pressure is gradually reduced according to the change in head from the cutting edges upward. Failure to keep the air pressure low enough while concreting has resulted in the concrete being blown out, under the cutting edge, and even coming up to the surface of the ground. Work is then suspended for at least 24 hours under air pressure, by which time the 5 feet of concrete will attain its permanent shrinkage. The air-lock is then taken off and concrete is dumped down the shaft to fill the space in the air chamber and some distance up the shaft. This concrete is made as wet as possible, while grout is used in some cases. If properly done, it will be found that the air has all been forced

up the vents and the grout from the concrete stand 6 to 20 feet up the vent pipes, thus indicating that the chamber is entirely filled. But this work must be done very quickly, continuously and with great care. It is the only reliable method.

ART. 192. JOINTS BETWEEN CAISSONS

There are several methods of making a joint between two caissons to prevent the flow of water between them. One method is by stock ramming, as applied on the Mutual Life Building foundations in 1900. The caissons were 8 feet wide, made of steel and filled with concrete. They were prevented from coming too close together by two 6- by 6-inch oak strips, spaced about 4 feet apart and held in place by 6- by 4-inch steel angles. Between the two caissons and these strips a 4-inch pipe was forced to rock, and pellets of clay were rammed down the pipe by an iron rod under the weight of a pile-hammer. This method exerts a high pressure and is capable of doing much damage if not carefully watched. For example, in trying to stop a leak in a dam, 500 cubic yards of masonry were cracked and lifted by the force of the clay driven through one pipe. The oak strips referred to above kept the clay from spreading and it was thus thoroughly compacted to hold back the water while the cellar was dug, and while 2 feet of brick from the inside face was placed between the ends of the caissons for a permanent water-tight wall. Forcing down grout instead of clay has also been tried, but did not prove as successful.

On the Commercial Cable Building, in 1896, the so-called half-moon joint was used for the first time. The steel caissons were 6 feet wide and so arranged that 4 feet of the end walls of each caisson could be removed after sinking. Behind these plates timber forms had kept the concrete back, leaving a semicircular opening, so that the two adjoining openings formed a shaft about 4 feet in diameter from the top to the bottom. Before removing the end sections, however, stock ramming was applied on each side, with the result that the clay filled not only the space between the caissons, but spread into the lot as far as 20 feet in extreme cases. After the sections were removed, the

vertical shaft was cleaned out and filled with concrete, making for the first time a continuous concrete wall all around the building to exclude the water.

On other work, no stock ramming was used, but a lock attached to a small shaft was concreted or bolted in place over the 3- or 4-foot circular shaft, and after the application of compressed air, the sand hogs closed the two openings in the shaft, working downward from the top and removing the material at the same time. This is often done by nailing short boards against upright timbers placed in the ends of the caissons before sinking. In one instance, these boards, not being strong enough or properly fastened, were blown out, allowing the ground to flow in and kill two men in the shaft or keyway. To close the opening between the caissons, by driving sheet-piling on each side before applying air, is quicker and cheaper than stock ramming but not nearly so effective (see Art. 126 for illustrations).

ART. 193. PLANT AND EQUIPMENT

The pipes for supplying compressed air are generally 4 inches in diameter, and there should be two from the caisson deck to the top to facilitate changing the connection as the cofferdam is built up. One 4-inch pipe is sufficient from the caisson to the compressors, with smaller pipes for high pressure to operate the locks where the old-fashioned locks are used. In winter these pipes should be placed in a box filled with manure to prevent freezing.

The compressors, electric-lighting and pumping plants are sometimes compactly arranged on a big float, although it often pays to locate them on shore alongside of a railroad track on account of coaling facilities. Where an old bridge is located next to the one under construction, it affords a good support for the pipe lines; or a light trestle may be built on piles to carry them; or the pipes may be laid on the river bottom, although this is not so desirable.

It is impossible to lay down any rigid rule for the size of plant required. It depends both on the number of caissons

and on the season of the year or climatic conditions. It always pays to have plenty of boiler capacity. For a bridge of fair size there should be two boilers of 150- and four of 80-horse-power capacity each. There should always be one more air compressor than is needed for constant service, to allow for repairs that will certainly be required. Nothing is so expensive on contract work as delay. A work of this magnitude probably requires three or four compressors with an aggregate capacity of 2500 to 4000 cubic feet of free air per minute.

In illustration of the effect of weather and location on the cost of work, two examples are given, in both of which the work extended over a year, including winter and summer. The first work was in the East, where about 20 caissons of medium size required 5000 tons of coal at a cost of $15,000. The second was in the West, where 5000 tons of coal were also required, but at a cost of $40,000. Although the number of caissons and the total yardage of caisson work were only about one-half as large as for the eastern location, yet on account of severe weather and higher price the coal cost over five times as much per cubic yard of caisson work. Both jobs were handled by the same contractor, and with the same staff and plant. This fact indicates one of the reasons why it is so difficult to compute the cost of pneumatic work in advance.

One of the best money-saving devices for a contractor who has a number of wooden caissons to build is a saw arbor run by compressed air or electricity. The time saved in cutting the large timbers to the right length, and in securing small timbers of the proper dimensions, pays for the machine in a short time. A good pipe-cutting machine with dies, etc., is also indispensable, as well as augers to bore holes for bolts and drift bolts and a hammer to drive them, both run by compressed air. An ample supply of the best stiff-leg and guy derricks, and necessary side tracks, wharves, cement and other storage buildings, will well repay the large outlay required. Cableways up to 1600 feet in span have been used to advantage in some cases, while in others they proved a source of loss.

ART. 194. AIR-LOCKS AND CONCRETE

One of the most important contrivances on a pneumatic caisson job is an air-lock, without which the work cannot be carried on. It consists of an air chamber with one door opening to the atmosphere and another into the shaft or working chamber. In the early caissons, the lock was placed below the shaft in the working chamber. This is an inconvenient and unsafe position for the lock, for if the caisson becomes too heavy there is danger of crushing the lock, also the lock has to be taken apart and removed before the shaft can be filled with concrete. The lock was probably put at the bottom to permit adding new sections to the shaft without removing the lock, and before the idea occurred to anyone of placing an additional door at the bottom of the shaft. This door is now used to prevent the escape of air when the lock is lifted off temporarily to add shafting. It is also useful in case of emergency.

Although it did not take long for the advantages of placing the lock at the top of the shaft to become apparent, the hoisting mechanism was placed inside of the lock. Accordingly, the bucket was lifted from the working chamber into the lock, the lower door closed and the material dumped through a side door or again lifted through a top door, thus requiring handling the material twice. This cumbersome and slow method is still used in Europe and occasionally in this country. The Mattson lock is an improvement on this method and has its proper place.

This arrangement was superseded by means of the modern locks which permit the bucket to be lowered into the working chamber, filled, hoisted out, emptied and returned to the working chamber without detaching it from the cable. The first lock to accomplish this saving of time and money had its top door in two horizontal halves, meeting over the center of the shaft and leaving a hole for a stuffing box 3 or 4 inches in diameter at the center of the joint. The stuffing box was so packed that the steel cable could pass through freely without allowing much air to escape. When the bucket was hoisted out of the lock, the stuffing box remained on the cable near the bale of the bucket. Later it was found by experiment that, by making

the hole in the doors only large enough for the cable to pass through, the loss of air was not sufficient to warrant the use of a patent stuffing box. Since there is no necessity for the cable to pass through the lower door of the lock when closed, the best form is a single round door slightly larger than the opening and hinged on one side. It is known as a flap door since it swings up against its seat where it is held by air pressure. A rubber gasket about ½ inch thick and 3 to 4 inches wide is usually attached to the door to prevent the escape of air between the door and its seat.

According to present practice, then, the derrick lowers the bucket into the lock, the upper doors close against the cable, after the lock is filled with air the lower door drops open by its own weight and the passage is clear for the bucket to be lowered into the working chamber. The bucket is filled and hoisted again into the lock, the lower door is swung up by levers on the outside, the air in the lock is allowed to escape, permitting the upper doors to be opened, and the bucket hoisted out and emptied. The entire cycle of operation for a half-yard bucket can be repeated 20 times an hour, a vast improvement over the older system.

Numerous patents have been taken out to get around the original one. One lock has a circular flap door at the top as well as at the bottom, the upper one having a slot extending from the center to the edge to permit the door to shut while the bucket is suspended in the lock. An additional contrivance covers the slot afterward. Another lock, more extensively used, has a circular top door so placed that the edge of the door is directly over the center of the shaft, permitting the hole for the cable to be located at the edge of the door instead of the center. This arrangement requires the lock tender to give the bucket or cable a slight push as it enters or leaves the lock. In a still later design, the cable passes through the door frame instead of the door. This is probably the best lock in use. Apparently, every practicable form of lock has been patented. All those described above have doors opening inward, so that when they are closed the air pressure holds them shut. This

is the only safe method, for the greater the pressure the tighter the door is held. However, locks have been built with upper doors closing from the outside and held shut by means of screws, etc. When the bucket is taken out of the lock, the door and stuffing box remain on the cable. But few of the locks were manufactured, as the patent was promptly bought by the owner of other patents.

Bucket locks are used extensively in concreting the working chamber as well as in excavating small caissons, but for large caissons having two shafts, a special concrete lock is used. It usually consists of an ordinary 3-foot shaft with a door in the bottom, and a cone above the lower door. The lock is placed on top of the shaft and has a hopper located above it. As soon as a yard or so of concrete has been dumped into the lock, the upper door is closed and the bottom one opened, allowing the mass to fall down the shaft into the working chamber. In this manner, concrete can be taken in about as fast as the men below signal for it. Often much of this work is done with no men in the working chamber part of the time.

All concrete for caisson work should be as non-porous as possible. The principal means toward this end consists in making the mixture of sand and cement in the proportion of one part, by volume, of cement to two parts of clean, sharp and coarse sand. Four, five or even more parts of stone or gravel to one of cement can be used for this mixture, provided it is made wet enough. A poorer mixture than one to two of cement and sand will not have the voids of the sand filled, while a wet 1-2-4 mixture will not usually have as much stone as can be safely covered. When dry concrete used to be employed, it was hard to get a 1-2-4 mixture properly rammed, but with wet concrete, the stone immediately disappears in the cement and sand, insuring good concrete without voids.

ART. 195. ALLOWABLE BEARING UNDER CAISSONS

The maximum pressure allowed on bed rock or good hardpan should be based on the strength of concrete, and should never exceed 15 tons per square foot. Good concrete, as indicated

by careful tests, will resist very much higher pressures, and so will bed rock and many kinds of hardpan; but in order to allow a reasonable factor of safety to cover imperfect work or material, even if such lapses occur only occasionally in the night, this pressure should not be exceeded.

Good sand on the surface, and not under a caisson, should not be loaded over 2 or 3 tons per square foot; but if it is under a caisson and 30 feet or more below the surface where it cannot be disturbed, it can safely be loaded to a maximum limit of 6 tons, although test loads have stood as high as 10 tons. In New York City, the hardpan varies from 2 to 30 feet in thickness, with 30 to 60 feet of quicksand above it, and sometimes from 2 to 40 feet of sand, boulders, etc., below it. It also varies in quality from a material resembling good concrete to that of loose sand. For clay and other materials, the variations are so great that no definite load should be specified until the local conditions of each case have been carefully examined and considered.

In one example, open concrete cylinders were sunk from 30 to 90 feet to beds of various grades of fine and coarse sand. The one which was apparently the most unfavorable was subjected to a test load of 10 tons per square foot, causing a settlement of about 1 inch, one-half of which was recovered upon removing the load. After a concrete viaduct to carry a railroad was built upon these cylinders, several of them began to settle, and continued until at the end of about a year the maximum was reached, some cases amounting to at least 6 inches. After that no further trouble occurred.

ART. 196. REMARKS ON UNDERPINNING

Since Chap. XVI on underpinning is so complete, but little remains to be added except to present conclusions. It is generally found to be more economical to use inclined shores, needles, or both, on light buildings, that is, on ordinary buildings up to six or seven stories high. For higher buildings, or where bed rock is easily accessible, the system patented by BREUCHAUD in 1896 can be depended upon to give good results, but

smaller diameters than 30 inches are not recommended, which permit sending men down to the bottom. The more recent patented systems, namely, MERRILL's telescopic method and THOMSON's vertical tunnel method, are fully described in Chap. XVI.

In the writer's experience, 16-inch cylinders have been jacked down under a six-story building until the weight of the old building was taken off the old foundations, and then, after the shoring was completed, these cylinders settled when an adjoining caisson was sunk, thus requiring the use of inclined shores after all. Later, when the Gillender Building was removed to give place to the Bankers' Trust Building, he witnessed the removal (in 1911) of 14-inch cylinders which had been sunk in 1897 under an adjoining building, and they were found to be filled with excellent concrete except within a few feet at the bottom, which was filled with sand. This observation probably accounts for the settlement just mentioned.

CHAPTER XIX

REFERENCES TO ENGINEERING LITERATURE

Art. 197. Literature on Foundations

Very few books have been published in this country which are devoted exclusively to the subject of foundations. In most cases the subject is treated in one or two chapters of a book, as indicated in the following list. The list is not complete but contains the most important works which should be accessible in college libraries. With a few exceptions, only American works are included. The dates of publication given are in most cases those of the first editions of the respective works.

American School of Correspondence. Cyclopedia of Architecture, Carpentry and Building. Chicago, 1907. Vol. 3 contains 22 pages on foundations.

Arthur, William.—Contractors' and Builders' Handbook. New York, 1911. Contains one chapter (18 pages) on foundations.

Baker, I. O.—Treatise on Masonry Construction. New York, 1889. The tenth edition contains four chapters on foundations, the titles of which are: Introductory; Ordinary Foundations; Pile Foundations; and Foundations under Water; covering about 18 percent of the volume. Bridge abutments and piers are treated in two additional chapters.

Buel, A. W., and Hill, C. S.—Reinforced Concrete. New York, 1904. The second edition contains 28 pages on reinforced-concrete footings and concrete piles.

Byrne, A. T.—Inspector's Pocket-Book. Materials and Workmanship in Construction. New York, 1892. The third edition contains 29 pages on foundations.

Corthell, E. L.—Allowable Pressures on Deep Foundations. New York, 1907. The entire book, containing 98 pages and 8 folding tables, is devoted to a record of pressure on deep foundations for 178 structures of different kinds located in different countries, as well as of the conditions in the respective cases.

Fiebeger, G. J.—Civil Engineering. New York, 1905. One chapter is devoted to foundations.

FOSTER, W. C.—Treatise on Wooden Trestle Bridges. New York, 1891. The fourth edition contains three chapters on pile-bents, pile-drivers, and concrete [pile] trestles. That on pile-bents includes some notes on pile driving.

FOWLER, C. E.—Engineering and Building Foundations. New York, 1920. This book supercedes Fowler's Subaqueous Foundations, published in 1914, Fowler's Ordinary Foundations, published in 1905 and Fowler's Cofferdam Process for Piers, published in 1898.

FREITAG, J. K.—Architectural Engineering. New York, 1895. Contains one chapter on the foundations of buildings, relating principally to steel-grillage and reinforced-concrete footings, with some data on foundation loads.

FRYE, A. L.—Civil Engineers' Pocket-Book. New York, 1913. Contains one section, 29 pages, on foundations.

GILBERT, G. H., WIGHTMAN, L. J., and SAUNDERS, W. L.—Subways and Tunnels of New York. Methods and Costs. New York, 1912. In one appendix, 12 pages are devoted to the sinking of pneumatic caissons for tall buildings in New York. Several other appendices give information on the use of compressed air for tunnel work, shaft sinking, etc., and on the equipment required.

GILLETTE, H. P.—Handbook of Construction Cost. New York, 1922. Includes data on cost of a number of types of foundations, including piling, cofferdams, caissons, piers and abutments.

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GILLETTE, H. P.—Handbook Cost Data for Contractors and Engineers. New York, 1906. Includes data on the cost of piles, drivers, making piles, driving piles, sawing off piles, pulling piles, blasting piles, puddle, a bridge foundation and cofferdam.

HARCOURT, L. F. VERNON.—Civil Engineering as Applied to Construction. London, 1902. Contains one chapter on foundations and piers of bridges, and another one on excavations, dredging, pile driving and cofferdams.

HILL, LEONARD.—Caisson Sickness and the Physiology of Work in Compressed Air. London, 1912.

HOOL, G. A.—Reinforced-Concrete Construction. Vol. 2. Retaining Walls and Buildings. New York, 1913. One chapter (49 pages) is devoted to foundations, including the bearing capacity of soils, shallow footings and concrete piles. Another chapter on retaining walls includes designs of their footings.

HOOL, G. A., and JOHNSON, N. C.—Handbook of Building Construction. New York, 1920. Prepared by a staff of 46 specialists. Contains about 65 pages on foundations.

HOOL, G. A., and KINNE, W. S.—Foundations, Abutments and Footings. New York, 1921. Besides treating the general subject of foundations, this book devotes some space to excavation methods and to legal provisions regarding foundations.

International Library of Technology. Scranton, 1905. Volume 52 contains section 18 (189 pages) on the simple types of footings and buttresses, and section 20 (70 pages) on shallow foundations and cantilever foundation girders.

KETCHUM, M. S.—Structural Engineers' Handbook. New York, 1924. Contains one chapter on piers and abutments, which includes some material on foundations.

KIDDER, F. E., and NOLAN, T.—Architect's and Builder's Pocket-Book. New York, 1884. The seventeenth edition contains one chapter (about 85 pages) on foundations and spread footings.

KIDDER, F. E.—Building Construction and Superintendence. Part I. Masons' Work. New York, 1896. Contains three chapters, respectively, on foundations on firm soils; on foundations on compressible soils; and on masonry footings and foundation walls, shoring and underpinning.

MAHAN, D. H.—Treatise on Civil Engineering. New York, 1873. Contains two chapters on foundations of structures on land, and in water, respectively. This work is out of print.

MERRIMAN, M., Editor-in-Chief.—American Civil Engineer's Pocket-Book. New York, 1911. Contains 35 pages on foundations on land and under water; and some other articles on foundations of reinforced concrete, on shafts and borings, etc.

MITCHELL, C. F.—Building Construction. London, 1913. (Seventh edition.) Contains one chapter on foundations, including piles, wall and column footings, drainage, and shaft sinking and trenching.

PATTON, W. M.—Practical Treatise on Foundations. New York, 1893. In the second edition about 50 percent of the book is devoted to the subject of foundations proper. In the first edition, the corresponding percentage was only 27.

PATTON, W. M.—Treatise on Civil Engineering. New York, 1895. Contains one chapter on foundations and foundation beds.

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REID, H. A.—Concrete and Reinforced-Concrete Construction. New York, 1907. Contains one chapter on foundations, devoted practically to shallow footings and reinforced-concrete piles.

RICHEY, H. G.—Building Foreman's Pocket-Book and Ready Reference. New York, 1909. Contains 8 pages on piles for foundations.

RICHEY, H. G.—Handbook for Superintendents of Construction, Architects, Builders and Building Inspectors. New York, 1905. Contains 25 pages on pile foundations and shallow footings.

SPALDING, F. P.—Masonry Structures. New York, 1921. Contains one chapter on foundations.

TAYLOR, F. W., and THOMPSON, S. E.—Treatise on Concrete, Plain and Reinforced. New York, 1905. The second edition contains one chapter (20 pages) on foundations and piers, treating particularly of single and combined footings of reinforced concrete and of concrete piles.

TAYLOR, F. N.—Manual of Civil Engineering Practice. London, 1911. Contains one chapter on foundations and pile driving.

TRAUTWINE, J. C., J. C. JR., and J. C. 3rd.—Civil Engineers' Pocket-Book. New York, 1872. The nineteenth edition contains one section of 18 pages on foundations.

WADDELL, J. A. L.—Bridge Engineering. New York, 1916. This valuable treatise on bridge engineering contains eight chapters on foundations: Foundations in General, 9 pages; Cofferdams, 8 pages; Open-Dredging Process, 17 pages; Pneumatic Process, 10 pages; Piles and Pile Driving, 12 pages; Piers, Pedestals, etc., 38 pages; Borings, 16 pages; and Quantities for Piers, Pedestals, Abutments, etc., 48 pages.

WHEELER, J. B.—Elementary Course of Civil Engineering. New York, 1876. Contains two chapters on foundations on land and in water, respectively. Out of print.

WHITE, LAZARUS—The Catskill Water Supply of New York City. New York, 1913. One chapter gives descriptions of borings and sub-surface investigations, and another one of exploration for the Hudson River crossing, including 49 pages in all.

WILLIAMS, C. C.—The Design of Masonry Structures and Foundations. New York, 1922. Contains four chapters on foundations, piers and abutments.

The following valuable monographs on important bridges and their foundations have been published in book form. They deserve study by engineers with reference to the historical development of American foundation practice.

CLARKE, T. C.—The Quincy Bridge. New York, 1869. Three chapters and an appendix are devoted to the physical characteristics of the Mississippi River, a description of the substructure and foundations, specifications and classified cost. The foundations and equipment for construction are illustrated by 12 plates. The open caissons, and coffer-dams with removable sides on grillage, were used for pile foundations, protected from scour by loaded timber cribs

CHANUTE, OCTAVE, and MORISON, GEORGES.—The Kansas City Bridge. New York, 1870. Four chapters give the regimen of the Missouri River, the foundations, masonry and classified cost of the work; an appendix

gives tables showing the progress of sinking a pier, with soundings, weights, etc.; while 6 plates illustrate foundation works, piers and equipment. Four piers were founded on bed rock with open timber caissons having dredging wells, while two piers were founded on piles.

WOODWARD, C. M.—History of the St. Louis Bridge. St. Louis, 1881. Six chapters are devoted to the deep pneumatic foundations for the two river piers and east abutment, the physiological effects of compressed air, computations on the stability of the piers, and on classified costs. Two other chapters relate to the west abutment which required a cofferdam to be built under extraordinary difficulties, to financial and engineering considerations relating to preliminary and final foundation plans and to sinking by the pneumatic process. The substructure and foundations are illustrated by 15 plates of plans and views.

MORISON, GEORGE S.—Plattsmouth Bridge, 1882; Bismarck Bridge, 1884; Blair Crossing Bridge, 1886; New Omaha Bridge, 1889; Rulo Bridge, 1890; Sioux City Bridge, 1891; Nebraska City Bridge, 1892; Cairo Bridge, 1892; Bellefontaine Bridge, 1894; Memphis Bridge, 1894. These reports give the most complete information about pneumatic foundations of any that have been published. The kinds of data given are indicated by the report on the Bellefontaine bridge. The general description includes the trestle approach on piles, the classified cost of each pneumatic foundation in detail and the cost and quantity of masonry in the piers. In appendices are given a record of sinking the caissons with elevations, immersion, weights, air pressure and skin friction; the time, costs and materials used in foundations; and the specifications for masonry. The plates show elevations and plans of the piers; detail drawings of the caissons; a diagram giving the rate of progress in sinking caissons; and a water-gage record. In most cases a record of the preliminary borings is also given. In the first two reports, however, the weights and skin friction for the caissons, computed daily while sinking, are not given.

HUTTON, W. R.—The Washington Bridge. New York, 1889. In 20 pages the text gives a brief description of the substructure, the specifications for the masonry, a table of diamond drill borings and a record of sinking the pneumatic caisson. The illustrations include 4 plates on foundations besides 20 on masonry.

BOLLER, A. P.—The Thames River Bridge. New York, 1890. The text gives a record of soundings and borings, a description of foundations, weights and settlement of piers. Three plates have illustrations on foundations. Four of the piers are supported by pile foundations, the upper parts of the piles being protected by timber cribs sunk into holes previously dredged. The masonry was built in cofferdams with removable sides on grillage, and sunk to bearing on the piles. After removing the cofferdams, a filling of sand and gravel was placed around the piers and in the cribs.

NOBLE, ALFRED, and MODJESKI, RALPH.—The Thebes Bridge. Chicago, 1907. The description of the substructure is supplemented by 14 plates and several half-tone views. The river piers were founded by the pneumatic process, and one of the shore piers by means of an open caisson of reinforced concrete. The specifications for the substructure are given in an appendix.

Cambridge Bridge Commission. Report of the Commission and of the Chief Engineer (WILLIAM JACKSON) upon the Construction of the Cambridge Bridge. Boston, 1909. The description of the substructure and the analysis of its cost are supplemented by 12 views, 11 folding plates and 6 folding schedules of expenditures. The piers and abutments are all supported on pile foundations.

MODJESKI, RALPH.—The Vancouver-Portland Bridges. Chicago, 1910. The description of the substructure is illustrated by 23 plates. The specifications are given in an appendix. This report includes the Washington Channel Bridge over the Columbia River, Shaw's (or Hayden's) Island Viaduct, the Oregon Slough Bridge, and the Willamette River Bridge. The piers of the bridges over the two rivers were founded by the pneumatic process, while the abutments and the remaining piers have pile foundations.

A large number of selected references to engineering periodicals and the proceedings or transactions of engineering societies are given in the following articles. They are intended for the benefit of those who desire to study any topic more extensively or in greater detail than the limits of this volume allow for the descriptions and illustrations given in the text. No attempt is made to refer to all the engineering periodicals published in this country, nor to include every reference that may be found in the periodicals selected. It will be observed that the titles of the following articles in this chapter correspond to those of preceding chapters. The authors will appreciate information regarding errors discovered in the references.

It is a valuable exercise for the student to compare the general arrangement and details of construction for any given type of structure relating to foundations, as designed by different engineers, and to note which features constitute the essential elements of that type, and which ones are dependent merely upon local conditions and therefore subject to more or less variation. To make the results of such studies readily available

for future reference, they should be placed upon separate sheets of paper and filed in accordance with a suitable classification of subjects.

ART. 198. TIMBER PILES AND PILE DRIVING

TIMBER PILES.—Life of Different Kinds of Timber Piles. Report of Committee and Discussion. Proc. Assoc. Ry. Supts. B. & B., 1899, v. 9, p. 50. Calculating the Cubical Contents of Piling. Eng. News, v. 54, p. 170, Aug. 17, 1905. Table prepared by E. O. Faulkner of Atchison, Topeka and Santa Fé Railway. See Richey's Building Foreman's Pocket-Book, pp. 474, 475. Bark Left on Underground Part of Creosoted Piles. Eng. News-Rec., v. 89, p. 159, July 27, 1922.

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ART. 203. BOX AND OPEN CAISSONS

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Art. 204. Pneumatic Caissons for Bridges

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ART. 205. PNEUMATIC CAISSONS FOR BUILDINGS

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for a Large Steel Cage Office Building on Broadway, New York. Eng. Rec., v. 49, p. 284, Mar. 5, 1904. Foundations of the Rogers Building, New York. Eng. Rec., v. 49, p. 362, Mar. 19, 1904. Auxiliary Pneumatic Caisson Work for the Bank of the State of New York. Eng. Rec., v. 48, p. 245, Aug. 29, 1903; Eng. Rec., v. 46, p. 242, Sept. 13, 1902. Foundations of the Gillender Building. Eng. Rec., v. 35, p. 140, Jan. 16, 1897; Eng. News, v. 37, p. 13, Jan. 7, 1897. Sinking Extensive Caisson Foundations for a St. Louis Hotel. Eng. News, v. 77, p. 458, Mar. 22, 1917.

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CONCRETE CAISSONS.—Foundations for the Morgan Building, New York City. Eng. News, v. 71, p. 70, Jan. 8, 1914. Equitable Building Foundations. Eng. Rec., v. 69, p. 448, Apr. 18, 1914. Largest Pneumatic-Caisson Job in Canada. Eng. Rec., v. 70, p. 398, Oct. 10, 1914. Pneumatic Caissons Sunk through Moving Ground. Eng. News-Rec., v. 82, p. 1160, June 12, 1919. Deep Substructure of Assay Office Built in Quicksand by Caisson-Inclosure Method. T. Kennerd Thomson. Eng. News-Rec., v 84, p. 165, Jan. 22, 1920. Caisson Cofferdam Foundation with Special Bracing. T. Kennerd Thomson. Eng. News-Rec., v 88, p. 914, June 1, 1922.

Art. 206. Pier Foundations in Open Wells

Open Wells with Sheet-Piling.—Construction of the New Plaza Hotel, New York City. Eng. Rec., v. 54, p. 553, Nov. 17, 1906. Difficult Foundations of the Hoffman House Extension. Eng. Rec., v. 55, p. 296, Mar. 2, 1907. Deep Open Excavation in Quicksand. Eng. Rec., v. 64, p. 769, Dec. 30, 1911. Excavating Caissons Hydraulically at St. Louis. Eng. Rec., v. 66, p. 262, Sept. 7, 1912. Foundations of the Bamberger Building. Eng. Rec., v. 64, p. 456, Oct. 14, 1911. Foundations of the Kinney Building, Newark, N. J. Eng. Rec., v. 66, p. 445, Oct. 19, 1912. Deep Open Pits for Foundation Piers. Eng. Rec., v. 67, p. 158, Feb. 8, 1913. Deep Foundation Pits in Quicksand. Eng. Rec., v. 67, p. 469, Apr. 26, 1913. Cylinder Pier Foundations Laid Inside Sheet-Pile Wells. Eng. News, v. 76, p. 289, Aug. 17, 1916; Eng Rec., v. 75, p. 510, Mar. 31, 1917.

Open Wells with Sheeting.—Foundations for the City Hall at Kansas City. Eng. Rec., v. 25, p. 292, 329, 403, Apr. 2, 1892; May 14, 1892. Chicago Foundations. Eng. Rec., v. 52, p. 131, July 29, 1905. Development of Deep Building Foundations, Chicago. Eng. News, v. 52, p. 560, Dec. 22, 1904. Foundation Work on the Cook County Building, Chicago. Eng. Rec., v. 53, p. 800, June 30, 1906. Steel-Piling Foundations. Eng. Rec., v. 53, p. 246, Mar. 3, 1906. Extension Ribs and Jacks for Caissons and Trenches. Eng. News, v 56, p. 117, Aug. 2, 1906. Foundations of the Northwestern Railway Terminal, Chicago. Eng. Rec., v. 59, p. 595, May 8, 1900. Foundations for the New City Hall in Chicago. Eng. Rec., v. 59, p. 745, June 12, 1909. Piers for Drawbridge over the Calumet River. Eng. Rec., v. 67, p. 208, Feb. 22, 1913. Chicago Foundations. Technograph No. 19, p. 5, 1904–05. Foundations in Chicago. Jour. W. Soc. Engrs., v. 10, p. 687, 1905. Multiple-Spool Hoist for Foundation Work. Eng. News, v. 65, p. 133, Feb. 2, 1911. Substructure of the Lake Street Bascule Bridge in Chicago Eng. News, v. 74, p. 934, Nov. 11, 1915. Foundations in Quicksand and Watery Gravel. Eng. News-Rec., v. 83, p. 414, Aug. 28, 1919.

Grouting Process.—Cofferdam without Timber or Iron. Eng. News, v. 25, p. 249, Mar. 14, 1891; Trans. Am. Soc. C. E., v. 24, p. 234, Mar., 1891. New Process for Dealing with Quicksand. Eng. News, v. 27, p. 420, Apr. 28, 1892. Making Concrete Foundations in Quicksand. Eng. News, v. 31, p. 533, June 28, 1894. Grouting the Foundations of the Merrimac River Bridge. Eng. Rec., v. 50, p. 218, Aug. 20, 1904. Grouting Foundations for a Bridge over the Danube River at Ehingen. Eng. News, v. 47, p. 35, Jan. 9, 1902. Grouting Concrete Viaduct Piers at Riverside, Cal. Eng. Rec., v. 52, p. 283, Sept. 9, 1905. Improved Methods of Constructing Foundations under Water. Trans Am. Soc. C. E., v. 29, p. 639, 1893; Trans. Am. Soc. C. E., v. 30, p. 579,

Dec., 1893. Tests of Grouting Gravel in River Beds. Eng. News, v. 69, p. 979, May 8, 1913. Grouting as a Method of Engineering Construction. Eng.-Contr., v. 48, p. 21, Jan. 13, 1915.

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ART. 207. BRIDGE PIERS

GENERAL.—Dimensions of Masonry Piers. Street Ry. Jour., v. 28, p. 398, Sept. 15, 1906. Concrete Piers. Ry. Age Gaz., v. 46, p. 165, Jan 22, 1909. Classified cost. Design and Construction of High Bridge Piers. Eng. News, v. 53, p. 548, May 25, 1905. Valuable article showing method of design, also shows examples of solid piers and hollow pivot piers. Mingo Bridge Approaches. Eng. Rec., v. 49, p. 789, June 25, 1904; Eng. Rec., v. 50, p. 27, July 2, 1904. Gives excellent description of methods of construction, building forms, etc. Concrete Bridge Piers. Eng. News, v. 30, p. 296, Oct. 12, 1893. Early use of all-concrete piers. Stability of Stone Structures. Trans. Am. Soc. C. E., v. 8, p. 238, Sept., 1879. Concrete Piers. Trans. Am. Soc. C. E., v. 29, p. 622, Sept., 1893; Trans. Am. Soc. C. E., v. 30, p. 567, Dec., 1893. Substructure of Piscataquis Bridge and Analysis of Concrete Work. Trans. Am. Soc. C. E., v. 61, p. 377, Dec., 1908. Distribution of Pressure on Piers. Eng. Mag., v. 12, p. 869, Feb., 1897. Design of Bridge Foundations. Eng. Rec., v. 38, p. 376, Oct. 1, 1898. Bridge Construction. Trans. Assoc. C. E., Cornell University, v. 1, p. 5, Apr., 1893. Bridge Work on the Kansas City, Pittsburgh and Gulf Railway. Eng. News, v. 40, p. 114, Aug. 25, 1898. Construction of Substructures and Foundations within a Radius of 60 Miles of Pittsburgh. E. K. Morse. Proc. Engr's. Soc. W. Pa., v. 27, p. 1, Feb., 1911. Uplift or Buoyancy. Proc. Am. Ry. Eng. Assoc., 1916, v 17, p. 229. Report on Typical Designs of Foundations for Piers, Abutments, Retaining Walls and Arches. Proc. Am. Ry. Eng. Assoc, 1917, v. 18, p 854; 1918, v. 19, p. 731. Obstruction of Bridge

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Solid Piers.—Saybrook Bridge on the Connecticut River. Eng. Rec., v. 65, p. 186, Feb. 17, 1912. Stone-masonry piers on pile and timber grillage foundations. Substructure of a Double-Track Railroad Bridge at Peoria, Ill. Eng. Rec., v. 62, p. 105, July 23, 1910. Reinforced with rods. Copper River Bridge Piers. Eng. Rec., v. 61, p. 642, May 14, 1910. Starling heavily reinforced with old rails. Piers of the Miles Glacier Bridge. Eng. Rec., v. 61, p. 559, Apr. 23, 1910. Heavily reinforced with rails against ice pressure. Bridge Piers on the Guelph and Goderich Railway Eng. Rec., v. 57, p. 77, Jan. 18, 1908. Piers of the Columbia River Bridge. Eng. News, v. 66, p. 301, Oct. 5, 1911. Piers of the Cantilever Bridge over the Ohio River at Beaver, Pa., Pittsburgh and Lake Erie Railroad. Eng. News, v. 63, p. 500, May 5, 1910; Proc. Engr's. Soc. W. Pa., v. 26, p. 1, Feb., 1910. Piers of the McKinley Bridge across the Mississippi River at St. Louis, Mo. Eng. News, v. 63, p. 9, Jan. 6, 1910. Large Concrete Pier. Eng. News, v. 53, p. 330, Mar. 30, 1905. The Mississippi River Cantilever Bridge at Thebes, Ill. Eng. News, v. 53, p. 470, May 11, 1905; Eng. Rec., v. 51, p. 263, Mar. 4, 1905. New Westminster Bridge over the Fraser River, British Columbia. Eng. News, v. 53, p. 611, June 15, 1905; Eng. Rec., v. 49, p. 679, May 28, 1904. High Concrete Piers for Railway Bridge across Stone's River Tennessee Central Railway. Eng. News, v. 47, p. 251, May 27, 1902. Cumberland Extension of the Western Maryland Railroad. Eng. News, v. 51, p. 304, Mar. 11, 1905. Reinforced-Concrete Piers of the Gilbertsville Bridge. Eng. Rec., v. 51, p. 265, Mar. 4, 1905; R. A. Gaz., v. 30, p. 31, July 14, 1905; Eng. News, v. 53, p. 548, May 25, 1905. Masonry Construction for the Blackwell's Island Bridge. Eng. Rec., v. 49, p. 307, Mar. 5, 1904; R. R. Gaz., v. 36, p. 319, Apr. 29, 1904. Substructure of the Cairo Bridge. Eng. News, v. 25, p. 122, Feb. 7, 1891; Morison's, "The Cairo Bridge." New Cornwall Bridge Piers. Eng. Rec., v. 40, p. 643, Dec. 9, 1899. An Artistic Bridge Pier, Eng. Rec., v. 70, p. 242, Aug. 29, 1914.

Hollow Piers.—Tall Reinforced-Concrete Bridge Pier. Eng. Rec., v. 62, p. 160, Aug. 6, 1910. St. Louis Municipal Bridge Substructure. Eng. Rec., v. 62, p. 427, Oct. 15, 1910; Eng. News, v. 65, p. 320, Mar. 16, 1911. Hollow Concrete Piers on the Louisville and Nashville Railroad. Ry. Age Gaz., v. 55, p. 146, July 25, 1913. Design of the Broadway or Sparkman Street Bridge, Nashville, Tenn. Eng. News, v. 62, p. 570, Nov. 25, 1909; Eng. News, v. 61, p. 199, Feb. 25, 1909. Substructure of the Mingo Bridge. Eng. Rec., v. 48, p. 393, Oct. 3, 1903. Monongahela Bridge Piers. Eng. Rec., v. 47, p. 2, Jan. 3, 1903. The Gunpowder and Bush River Bridges. Eng. News, v. 68, p. 144, Aug. 9, 1913; Eng.-Contr., v. 41, p. 195, Feb. 11, 1914. Railway Uses of Concrete.

Bull. 238, Am. Ry. Eng. Assoc., p. 74, Aug., 1921. Hollow Concrete Bridge Piers Built Like Chimney. Eng. News-Rec., v. 93, p. 757, Nov. 6, 1924.

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METAL-SHELL CYLINDER PIERS.—Cylinder-Pier Bridges, Central and North Western Railway. Eng News, v. 68, p 748, Oct 24, 1912. Cylinder Piers of the Norfolk and Western Bridge No. 5, Elizabeth River, Norfolk, Va. Eng. News, v 61, p 620, June 10, 1909. Modern Highway Bridge Construction. Eng. News, v. 64, p. 209, Aug. 25, 1910. Substructure of the Dumbarton Point Bridge Eng. Rec., v. 62, p. 172, Aug. 13, 1910; Trans Am Soc. C. E., v. 76, p. 1572, Dec., 1913. Tensas River Bridge. Eng. News, v. 13, p. 386, June 20, 1885. Bridge Foundations in Nova Scotia. Trans. Am. Soc. C E., v. 29, p 622, Sept., 1893, Trans. Am. Soc. C. E., v. 30, p. 567, Dec., 1893. Design of Concrete Piers with Metal Shells Eng News, v. 48, p. 379, Nov. 6, 1902. Cylinder Piers of the New Portland Bridge. Eng. Rec., v. 53, p. 252, Mar. 3, 1906. Steel Wharves at Manila. Eng. Rec., v. 53, p. 741, June 16, 1906. Dunsbach Ferry Bridge. Eng. News, v. 44, p. 54, July 20, 1901. Greenfield Street Railway Bridge, Greenfield, Mass. Eng. Rec., v. 49. p. 462, Apr. 9, 1904. Building Bridge Piers in the Mississippi River. Eng. News, v. 77, p. 382, Mar. 8, 1917.

REINFORCED-CONCRETE CYLINDER PIERS.—Piers for Bridge over the St. Croix River at Hudson, Wis. Eng. Rec., v. 69, p. 192, Feb. 14, 1914. Lift Bridges over the Buffalo River. Ry. Age Gaz., v. 54, p. 197, Jan. 31, 1913. Reinforced-Concrete Piers for a Bridge at Stakeford, England. Eng. News, v. 63, p. 193, Feb. 17, 1910. Open-Well Piers. Eng. News-Rec., v. 88, p. 180, Feb. 2, 1922. High Cylinder Piers. Eng.-Contr., v. 41, p. 9, Jan. 7, 1914.

TIMBER PIERS.—Pile Piers, Crib Piers and Pile and Crib Piers. Eng.-Contr., v. 44, p. 252, Sept. 29, 1915. Pile and Concrete Piers for Missouri River. Eng. News-Rec., v. 84, p. 824, Apr. 22, 1920. Sheathed

Pile Piers for Bridges. Eng. News-Rec., v. 83, p. 421, Aug. 28, 1919. Piers. Eng. News-Rec., v. 84, p. 287, Feb. 5, 1920.

PIVOT PIERS.—Substructure of the East Haddam Bridge. Eng. Rec., v. 66, p. 630, Dec. 7, 1912. Substructure of the St. Louis River Bridge. Eng. Rec., v. 65, p. 582, May 25, 1912. Pivot Pier of the Chelsea Bridge North. Eng. News, v. 68, p. 138, Aug. 2, 1913. Pivot Pier of the Gilbertsville Bridge. Eng. Rec., v. 51, p. 265, Mar. 4, 1905; Eng. News, v. 53, p. 548, May 25, 1905. Draw Foundation Pier for Charlestown Bridge. Eng. Rec., v. 38, p. 186, July 30, 1898. Substructure of the Dumbarton Point Bridge. Eng. Rec., v. 62, p. 172, Aug. 13, 1910; Trans. Am. Soc. C. E., v. 76, p. 1572, Dec., 1913. The New Portland Bridge. Eng. Rec., v. 53, p. 252, Mar. 3, 1906. Pivot Pier of the Interstate Bridge, Omaha, Neb. Eng. News, v. 30, p. 410, Nov. 23, 1893.

ART. 208. BRIDGE ABUTMENTS

GENERAL.—Design of High Abutments. Eng. News, v. 55, p. 36, Jan. 11, 1906. Economical Concrete Abutment. Eng. News, v. 55, p. 296, Mar. 15, 1906. Heaving of Bridge Abutments by Frost in the Ground. Eng. News, v. 59, p. 260, Mar. 5, 1908. Designing Concrete Abutments for Steel Highway Bridges. Eng. News, v. 65, p. 190, Feb. 16, 1911; Eng. Rec., v. 63, p. 305, Mar. 18, 1911. Gives diagrams for estimating the amount of concrete and the cost of abutments. Abutments for a Reinforced-Concrete Girder Bridge at Stakeford, England. Eng. News, v. 63, p. 193, Feb. 17, 1910. Concrete Pedestal Bridge Abutments on the New York State Barge Canal. Eng. News, v. 64, p. 180, Aug. 18, 1910; Eng. Rec. v. 61, p. 154, Feb. 5, 1910. Design of Railway Bridge Abutments. J. H. Prior, Proc. Am. Ry. Eng. Assoc., 1912, v. 13, p. 1086. Discussion of Design and Specifications for a Reinforced-Concrete Bridge Abutment. Trans. Can. Soc. C. E, v. 21, p. 173, 1907. Report on Principles of Design of Plain and Reinforced Retaining Walls and Abutments. Proc. Am. Ry Eng. Assoc., 1917, v. 18, p. 857.

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